

DEVELOPMENT OF A MODEL CODE FOR DIRECT DISPLACEMENT BASED SEISMIC DESIGN

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1 INTRODUCTION

Research Line IV of the RELUIS (Rete di Laboratorie Univeristari Ingegneria Sismica) project was responsible for the development of displacement-based design guidelines, considering structural typologies commonly utilised in Italy. The main aim and product of the research line has been the formation of a model code for displacement-based seismic design. The background of this research holds an interesting place in the history of seismic design and recent developments provide several strong motives for the further development of displacement-based seismic design methods.

2 BACKGROUND AND MOTIVATION

Seismic design in current codes is based on force (and hence acceleration) rather than displacement, essentially as a consequence of the historical developments of an understanding of structural dynamics and, more specifically, of the response of structures to seismic actions and the progressive modifications and improvement of seismic codes worldwide.

In the first decades of the last century, after several major earthquakes, such as Messina (Italy, 1908), Kanto (Japan, 1925), Napier (New Zealand, 1932), and Long Beach (USA, 1933) the first design codes started being developed. These codes were essentially prescribing specific detailing and construction rules and in cases the application of some lateral inertia forces. Typically, and possibly in analogy with some kind of wind design, a value of about 10% of the building weight applied as a vertically distributed lateral force was required, regardless of building period.

This initial force-based approach has been essentially retained with the progressive increasing of understanding of the significance of structural dynamic characteristics, that lead to period-dependent design lateral force levels in most seismic design codes, and even when it became clear that many structures had survived earthquakes capable of inducing inertia forces many times larger than those corresponding to their structural strength, if a linear response was assumed.

This apparent inconsistency was explained after the first simple inelastic time-history analyses had been performed, and the concept of ductility introduced, to reconcile the anomaly of survival with inadequate strength. In the seventy's, relationships between ductility and force-reduction factor were developed, introducing the well known concepts of "equal displacement", "equal energy" and "equal force" approximations, that appeared to be appropriate to estimate the "real" structural response as a function of linear response and period of vibration of the structures.

Since then, ductility has been considered the fundamental parameter to estimate appropriate “force reduction factors”, to be used to determine the design lateral force levels. Much research effort was therefore directed to determining the available ductility capacity of different structural systems, performing extensive experimental and analytical studies to determine their safe displacement capacity. It is now clear that this approach is implicitly assuming displacement capacity, and not force capacity, as the basis for design. However, the design process is still carried out in terms of required strength in essentially all codes of practice around the world and displacement capacity, if directly checked at all, is only a final product of the design procedure.

This brief summary of the history of seismic design indicates that initially design was purely based on strength, or force considerations. However, as demonstrated by Priestley (1993, 2003) there are several conceptual drawbacks associated with the use of force-based methods in seismic design. With reference to the design of a RC building possessing three walls A, B and C, illustrated in Figure 1, a brief review of some of the problems with force-based design can be made.

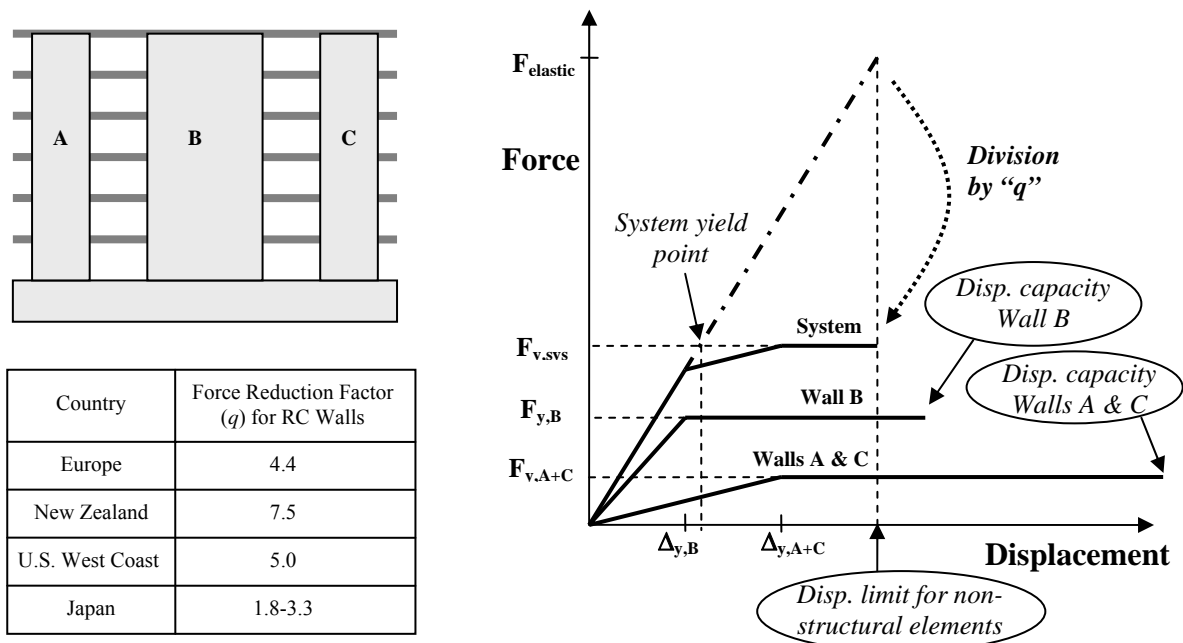


Figure 1. Response of a RC structure possessing walls of different length – used to highlight issues associated with force-based design.

The main issues associated with current force-based design methods, identified and discussed by Priestley (2003) in detail, can be shown to involve:

- *The use of ductility capacity dependent force-reduction factors.* In order to obtain design force levels, code methods divide elastic forces by a force-reduction factor which is set in proportion to the ductility capacity of the structure. However, the actual ductility demand for a structural system will typically be smaller than the ductility capacity of the structure. The right side of Figure 1 presents the force-displacement behaviour for the three walls (A, B and C) and the total system as it is displaced to a deformation limit required to control damage to non-structural elements. It is clear that in controlling non-structural deformations the system ductility is less than the ductility capacity of the long stiff Wall B and considerably less than the shorter flexible Walls A & C. Furthermore, as Walls A & C have considerably larger yield displacements

than Wall B, it is evident that the ductility capacity could not be developed in all the walls simultaneously. This point suggests that the use of force-reduction factors that are based on ductility capacity is inappropriate.

- *The force distributions predicted through the use of the elastic stiffness for analysis.* As mentioned above, the force-based approach makes a prediction of the elastic forces of the structure and uniformly reduces these by a behaviour factor to obtain inelastic design forces. However, because elements within the structure do not all yield at the same level of deformation, the elastic force distribution can be very different to the inelastic force distribution. This can be seen in Figure 1 by considering the elastic shear proportions that have developed when Wall B first yields. It is clear that the shear in Walls A & C is only one-quarter that for Wall B in the elastic state. However, at the design displacement of the system, it is clear that the proportion of shear in Walls A & C has now doubled to be 50% of that in Wall B. This point demonstrates that elastic analyses using the initial stiffness are inappropriate for predictions of inelastic force distributions.
- *The difficulty in defining the system ductility for mixed structural systems.* Current codes typically require that the behaviour factor for mixed structural systems be set equal to the lower of the two systems. However, this does not consider how the ductility demands and forces will develop for the combined structural system. For example, suppose that Wall B shown in Figure 1 only had a ductility capacity of three, such that at the design displacement, its ductility capacity had been reached. The system ductility at this displacement is obtained by dividing the design displacement by the yield displacement of the system, whereby the yield displacement of the system would typically be obtained through a bi-linear representation of the system response. Interestingly, it is evident that the system ductility demand is actually lower than the ductility capacity of the critical element. The amount by which it is lower depends on the mixed system considered and larger differences could be expected for different mixed systems, such as frame-wall structures. This point demonstrates that force reduction factors cannot be easily set for mixed structural systems.
- *The inter-dependency of strength and stiffness for certain structural types* (such as RC structures). It has been shown by Priestley (1998), Priestley and Kowalsky (1998) and Paulay (2002) that the yield curvature of RC sections is principally a function of the section geometry and yield strain of longitudinal reinforcement. Consequently, given that the cracked section stiffness is best defined using the secant stiffness to first yield as $EI = M_n / \phi_y$, (where E is the section modulus, I is the second moment of inertia, M_n is the section flexural strength and ϕ_y is the yield curvature) it is clear that the stiffness of a RC element will depend on the strength it is assigned. In other words, the initial stiffness of Wall B in Figure 1 for example, could be doubled by simply doubling the amount of longitudinal reinforcement in the section. As such, the stiffness is not purely a function of the section geometry and therefore in order to know the cracked elastic period of vibration of an RC structure, the flexural strength is required. As the force-based design procedure relies on the period of vibration in order to determine the required strength, this point shows that the design procedure cannot be easily implemented for RC structures.
- *The relationships used to relate elastic displacement to inelastic displacement response.* The force-based design approach estimates the inelastic displacement response based on the elastic displacement response. The relationship illustrated for the wall system within Figure 1 uses the equal-displacement rule, whereby the inelastic displacement is assumed equal to the elastic displacement. This is the

approximation adopted in Eurocode 8. In the United States, however, a different relationship is used whereby the inelastic displacement is typically approximated as being less than the elastic displacement, whereas in Japan the opposite occurs and an equal energy approach is used such that the inelastic displacement is estimated to be larger than the elastic displacement. In reality, the relationship between elastic and inelastic displacements should depend on the hysteretic properties of the structure being considered. Consider the behaviour of two moment-resisting frames with the same strength, stiffness and mass, different only in that one is formed with steel elements and the other with RC elements. As the initial period of the structures is the same, the equal-displacement rule would predict the same displacement for the two systems. However, it is well known that steel frames dissipate considerably more hysteretic energy than concrete frame systems and as such, one would expect that the inelastic displacement of the steel frame would be less than that of the concrete frame. This point shows that relationships between elastic and inelastic displacement should consider the hysteretic properties of the structure.

As these problems with force-based design have emerged and the importance of displacement has come to be better appreciated, attempts to modify and improve existing force-based approaches have been made. In the last decade, however, several researchers started pointing out the inconsistency associated with the use of force for design, proposing displacement-based approaches for earthquake engineering evaluation and design, with the aim of providing improved reliability in the engineering process by more directly relating computed response and expected structural performance.

A rather complete literature review of the subject is reported in *fib* Bulletin 25 (Calvi, 2003), where most displacement-based approaches proposed in the literature, are summarized, critically reviewed and compared, to favour code implementation and practical use of rational and reliable methods.

Comparison of different approaches, however, does not imply the production of a scale of merit, but rather an assessment of the relative ease or difficulty with which the design methods can be applied and any apparent limitations the methods may have. To compare the required strength obtained by each method and the performance of the methods in terms of predicted displacement or drift values for each case study and those obtained through time-history analysis may allow a future refinement of most proposals and a possible focusing towards the most efficient ones.

This objective has to be pursued discussing performance and hazard levels, fundamental principles to estimate structural response and displacement demand, proposed displacement – based design approaches, specific features of the description of seismic input, simple methods to assess the real displacement capacity of members and actual performance of different design approaches when applied to a number of different case study structures.

Recognising these considerations and in particular the observation that among the displacement-based approaches proposed in the literature, the direct displacement-based design (DDBD) method developed by Priestley, Calvi and Kowalsky (2007) amongst others, seems to be very promising and efficient, the goals of RELUIS research line IV were to:

- develop DDBD for different structural typologies commonly found in Italy;
- verify its applicability and effectiveness considering both simplified structural schemes as well as real case studies;
- verify its reliability running time history analyses of the designed structures and checking the structural response with the design expectations.

- verify its effectiveness applying force-based design to the same case studies and comparing the performance of the two methods.
- improve the method in those cases where the performed analyses have highlighted an unsatisfactory performance and re-run the analyses.
- obtain a set of displacement-based design guidelines enriched by the collection of the developed case studies to be used as reference and example.

In order to achieve these goals, a number of research units were needed and a clear research plan was required.

3 RESEARCH STRUCTURE

In order to achieve the numerous research goals, a number of Universities across Italy were contacted to participate in the project. Considering the background and expertise of the researchers involved, each University was designated as a research unit and was assigned a structural typology for which the DDBD methodology was to be developed, as shown in Table 1.

Table 1. Research Units for the project.

Research Unit	Research Topic	Research Institute
1	Principles, General Aspects, Actions	University of Pavia
2	RC Frames Structures	University of Bologna
3	RC Wall and Frame-Wall Structures	University of Ferrara
4	Pre-cast Reinforced Concrete Structures	University of Bergamo
5	Masonry Structures	University of Genova
6	Steel Structures	University of Napoli
7	Composite Steel-Concrete Structures	University of Benevento
8	Timber Structures	University of Trento
9	Bridge Structures	Politecnico di Milano
10	Isolated Structures	University of Basilicata
11	Foundation Structures	Politecnico di Milano
12	Retaining Structures	University of Perugia

Together with the allocation of structural typologies, a research programme, illustrated in Table 2, was set for the research units to follow. The time table of activities presented in Table 2 identified a series of activities (common to all the Units) divided by year. In the first year the main activities were the definition of the general and specific aspects of DDBD, selection of case study structures, and force-based design. In the second year the aim was to undertake displacement-based design and time history analyses of the case study structures selected in year 1, highlighting and discussing any problematic aspects. In the third and final year the main activities envisaged were to improve the method, re-run case studies comparing design predictions with non-linear time-history response, and finally define a set of guidelines for DDBD, producing a model code and commentary as well as a set of case-study examples.

4.1 Principles, Actions and General Aspects of Direct DBD

The main product of RELUIS Research Line IV has been the formation of a model code for Displacement-Based Seismic Design (Calvi and Sullivan 2009). This achievement was made considerably easier because of the release, during the course of the research, of a book on Displacement-Based Seismic Design by Priestley et al. (2007). This publication provided the various research units a clear understanding of the DDBD approach, and highlighted the state-of-the-art for the various structural typologies. In order to form the model code, however, there were still a number of considerations that had to be made in order to provide a relatively complete set of design recommendations.

The DDBD method aims to establish the level of strength that a structure requires in order that the displacements are within pre-defined limits during the design seismic event. To this extent, the exact intensity of the design event and the limits of displacement are inputs to the design procedure that can be selected by the designer (in consultation with the client or building owner) so that a suitable performance-based design solution can be achieved. Another aspect of the procedure is that rather than use a design acceleration spectrum, the approach utilises a design displacement spectrum. In the model code, the magnitude of design displacement spectrum has not been set as this will depend on the seismicity of the zone in which the structure is located. However, the form of the design displacement spectrum has been set, through consultation with members of the Research Unit 6 of the Project S5 (Cauzzi et al. 2009), to be that shown in Figure 2.

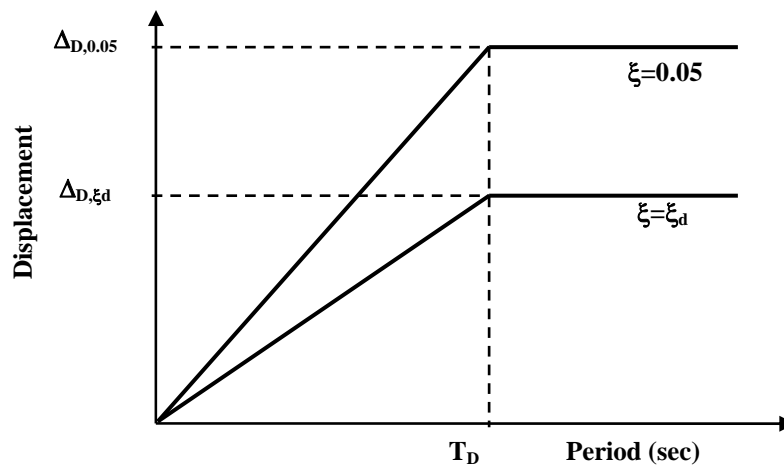


Figure 2. Form of the design displacement spectrum specified in the model code.

The shape of the displacement spectrum indicated in Figure 2 is very simplistic in that it only requires definition of a spectral displacement corner period, T_D , together with the magnitude of the displacement, $\Delta_{D,0.05}$, at the corner period (note that suitable values for such parameters were one of the objectives of Project S5; refer to report by Cauzzi et al. 2009). One initial concern with such a spectrum was that corresponding acceleration demands in the short period range are considerably higher than normal acceleration spectra would specify. To overcome this, the model code sets a limit on the design base shear, as indicated in Eq. (1), that corresponds to the plateau of the equivalent acceleration spectrum.

$$V_b = K_e \Delta_d + C \frac{P \Delta_d}{H_e} \leq 2.5 R_\xi \cdot PGA \cdot m_e + C \frac{P \Delta_d}{H_e} \quad (1)$$

Where V_b is the design base shear, K_e is the effective stiffness, Δ_d is the design displacement, C is a constant used to control P-delta effects, P is the total expected load and H_e is the effective height of the equivalent SDOF system. The limit to the base shear is set using R_ξ , the spectral reduction factor associated with the expected energy dissipation, the peak ground acceleration (PGA) expected at the site and m_e , the effective mass of the equivalent SDOF system. As such, the approach assumes that the elastic spectral acceleration plateau can be taken as 2.5 times the PGA. While such an approach may need refinement in the future to account for different soil types, this new proposal is clearly a simple but effective way of obtaining reasonable design base shear values for short period structures whilst maintaining the simplified form of the design displacement spectrum.

Another innovative inclusion within the model code relates to the required strength of a structure that possesses a design displacement greater than the corner displacement Δ_D . In the model code, the required effective period, T_e , is obtained using Eq.(2).

$$T_e = \frac{\Delta_d}{\Delta_{D,\xi}} \cdot T_D \quad (2)$$

Where T_D is the spectral displacement corner period (refer Figure 2) and $\Delta_{D,\xi}$ is the spectral displacement demand at this period for the anticipated level of equivalent viscous damping. Typically, designers will find that Eq.(2) yields effective period values less than T_D , since the design displacement of the system will typically be less than $\Delta_{D,\xi}$. However, in the event that Δ_d is greater than $\Delta_{D,\xi}$, Eq.(2) essentially linearly extrapolates the displacement spectrum ignoring the fact that the displacement demands plateau and do not develop the design displacement level. Considering the first mode response only, the designer might argue that because the spectral displacements plateau the structure should not require lateral strength to control the deformation demands to acceptable values. However, if the lateral strength is set to very low values such that the structural periods are very long, it is likely that higher mode deformations would become much more significant and performance limits states could be exceeded. As such, to ensure some mitigation of higher mode effects in a simplified manner, Eq.(2) is included in the model code. In addition however, in recognition of the fact that the 1st mode displacement demands are limited at long periods, a maximum value for the required effective stiffness, $K_{e,max}$, has been set through Eq.(3).

$$K_{e,max} = 4\pi^2 m_e / T_e^2 \frac{\Delta_{D,el}}{\Delta_d} \quad (3)$$

Where $\Delta_{D,el}$ is the corner spectral displacement demand for the elastic damping level, and the other symbols have been defined above. It is evident that Eq.(3) reduces the required base shear by the ratio of the maximum expected spectral displacement demand to the design displacement. While this reduction is not supported by rigorous theory and it is expected that improved procedures could be developed as part of future research, it does provide a simplified means of considering the spectral displacement plateau in design.

In addition to the design spectral shape, the design intensity for different performance objectives has been indicated by specifying the probability of exceedence of design ground motions as a function of the importance class of the structure at three different performance levels, as indicated in Table 3.

Table 3. Specified Probability of Exceedence for Different Structural Categories and Performance Levels.

Importance Class	Earthquake Design Intensity		
	Level 1	Level 2	Level 3
I	Not Required	50% in 50 years	10% in 50 years
II	50% in 50 years	10% in 50 years	2% in 50 years
III	20% in 50 years	4% in 50 years	1% in 50 years
IV	10% in 50 years	2% in 50 years	1% in 50 years

The three different performance levels, level 1, level 2 and level 3, correspond to a serviceability limit state, a damage control limit state and a collapse prevention limit state respectively. In order to achieve the desired performance, deformation limits have also been set for each limit state. Tables 4, 5 and 6 present the drift and strain limits set in order to control the performance of structural and non-structural elements. The research group has taken the initiative to set a number of strain limits for the first time, as guidance on strain limits for certain material types (e.g. soils and structural steel) could not be found in the literature. This fact perhaps emphasises the lack of importance that has traditionally been placed on the role of deformations for design. While effort has been made by Research Line IV to ensure that the limits indicated in these tables will provide the intended structural performance, it is expected that as the relationships between damage and deformation become better quantified (particularly for non-structural elements) the values indicated will be updated and the model code revised.

Table 4. Drift limits specified for different performance levels.

Drift Limit	Level 1	Level 2	Level 3
Buildings* with brittle non-structural elements	0.005	0.025	No limit
Buildings* with ductile non-structural elements	0.0075	0.025	No limit
Buildings* with non-structural elements detailed to sustain building displacements	0.010	0.025	No limit
Framed timber walls	0.01	0.02	0.03
RC Bridge Piers**	θ_v	0.03	0.04
Piers of Base Isolated Bridges	$2/3*\theta_v$	$2/3*\theta_v$	θ_v

*Base isolated buildings also fall into this category. **Optional – see code commentary

Table 5. Strain limits specified for structural elements for different performance levels.

Material	Level 1	Level 2	Level 3
Concrete comp. strain	0.004	Eq.(2.2) < 0.02	1.5Eq.(2.2)
Re-bar tension strain	0.015	$0.06\varepsilon_{su} < 0.05$	$0.09\varepsilon_{su} < 0.08$
Structural Steel Strain - Class 1 Sections, Flexural Plastic Hinges	0.010	0.025	0.04
Structural Steel strain - Class 2* & 3 Sections, Flexural Plastic Hinges	ε_y	ε_y	ε_y
Steel Brace Deformation Limits (see Eq. 2.3)	$\chi_{br}\varepsilon_y$	$0.25\mu_f\varepsilon_y$	$0.5\mu_f\varepsilon_y$
Reinforced Masonry comp. strain	0.003	Eq.(2.2) < 0.01	1.5Eq.(2.2)
Uneinforced Masonry comp. strain	0.003	0.004	0.004
Timber tension strain	$0.75\varepsilon_v$	$0.75\varepsilon_v$	$0.75\varepsilon_v$
Base Isolated Structures	As per section 2.4.3 of code		

Table 6. Strain Limits for Soils at Different Design Intensity Levels.

System	Level 1	Level 2	Level 3
Soil strain, γ	γ such that $G/G_{\max} \geq 0.80$	γ such that $G/G_{\max} \geq 0.20$	γ such that $G/G_{\max} \geq 0.20$
Retaining Walls Distant from Structures	γ such that $G/G_{\max} \geq 0.40$	γ such that $G/G_{\max} \geq 0.20$	γ such that $G/G_{\max} \geq 0.20$

In addition to maximum drift and strain limits, the model code has also proposed residual drift limits as shown in Table 7. However, except for the case of retaining structures (see Section 4.10) the residual drift limits are not obligatory in the code owing to the current difficulty that exists in proving compliance for normal structural solutions. The magnitude of the residual drifts were set considering limits typically set on the height over displacement ratios for other load types such as wind.

Table 7. Proposed Residual Drift Limits.

Drift Limit	Level 1	Level 2	Level 3
Building Structures	0.002	0.004	No limit
Bridge Structures	0.002	0.004	No limit
Retaining Walls distant from Structures	0.005	0.010	No limit

In the seismic design of a structure there are many complex phenomena that should be considered. However, in order to ensure a practical set of guidelines, efforts have been made to maintain simplified procedures whenever possible. For example, in order to control P-delta effects, special analyses are not specified in the code and instead the design base shear includes a single p-delta component which can be seen on the right side of Eq.(1). This term is in line with the recommendations of Priestley et al. (2007) and accounts for different hysteretic types through a p-delta constant C . Values of C of 1.0 and 0.5 are specified for steel and concrete structures respectively, thereby enabling designers to quickly account for p-delta effects in calculating the required design strength.

Another example is the simple procedure that has been provided to account for the twist that can develop for structures that are irregular in plan. Twist of a structure tends to increase the deformations on one side of a structure, and if not accounted for peak drifts would exceed the design drifts in this region. To account for such behaviour in a simplified manner, the design displacement of a system is multiplied by a torsion factor, ω_t , as shown in Eq.(4).

$$\Delta_d = \omega_t \frac{\sum_{i=1}^n (m_i \Delta_i^2)}{\sum_{i=1}^n (m_i \Delta_i)} \quad (4)$$

The basic reduction provided by ω_t should take account of the additional deformations that occur at the perimeter of the structure relative to the centre, due to torsion. As such, the model code states that ω_t is the ratio of maximum storey drift at the centre of mass to the maximum storey drift at the perimeter of the structure. Research by Beyer et al.(2008) indicated that the displacements of the perimeter of a plan-irregular building are typically not greater than 10% the centre of mass displacements if the strength eccentricity (see 7.1.3) is zero. Effectively, the findings of Beyer et al. (2008) indicate that the inelastic twist of a structure can be controlled by designing for zero strength eccentricity. As such, the model code states that the drift components required to define ω_t can be obtained from elastic analyses if the structure is provided with zero strength eccentricity. Alternatively, the model code suggests (in the

Commentary) that torsion be allowed for through advanced analyses or following the recommendations provided by Priestley et al. (2007).

By monitoring the work done by researchers external to the RELUIS project, it has been possible to incorporate several new research findings that are useful for displacement-based design. For example, a set of simplified yield curvature expressions are provided in Annex 1 of the model code and these include new expressions for the yield curvature of T and C-shaped RC walls developed by Smyrou et al. (2008) and Beyer et al. (2008) respectively. Another example is the set of recommendations provided to consider floor diaphragm flexibility and consider diaphragm design forces that are based on the findings and recommendations of Rivera (2008).

The model code has not only addressed the need for a coherent set of displacement-based guidelines for structures, but has also recognised that the success of a seismic design solution depends equally on an adequate capacity design method. As such, in the final chapter of the model code a number of alternatives for capacity design have been specified. The main uncertainty for capacity design is the effect that higher modes have on member forces. For example, Priestley and Amaris (2002) demonstrated that higher mode forces in RC wall structures are not effectively limited by the development of a plastic mechanism, as current code modal response spectrum methods assume. The general capacity design approach that is permitted for all structural types in the model code is to undertake non-linear time-history analyses of the structure, with strengths of plastic hinge regions set in line with the DBD findings. However, given the high level of expertise required to undertake non-linear time-history analyses, two alternative procedures have also been included. The first is a modified modal superposition approach in which first mode forces are set considering the actual design strengths provided and higher mode force components are estimated by undertaking modal analyses using structural models characterised with the effective stiffness of the structure at peak displacement response. While this approach has not been verified for all structural types, Priestley et al. (2007) argue that it provides the most consistent results of the modal methods available. The second, more simplified alternative to non-linear time-history analyses is to use an approximate method that is provided for RC frame structures, RC wall structures and RC frame-wall structures. An approximate but rather involved capacity design method is also given for isolated structures in an Annex of the model code. The approximate methods include empirical relations obtained by reviewing the results of a large number of non-linear time-history analyses of such structures. While the capacity design procedures provided in the model code do provide practitioners with a useful means of providing a good level of protection against undesirable inelastic mechanisms, it is considered that refined capacity design procedures, applicable to a wider range of structural typologies, could be developed as part of future research.

4.2 Reinforced Concrete Structures

As seen in Table 1, a number of different research units were formed to study RC structures with work done on reinforced concrete frame, wall, frame-wall and pre-cast structures. A relatively large amount of literature already existed for the displacement-based design of RC structures, and therefore much of the work done in this field was to test and refine the existing methodologies considering structural configurations common in Italy.

The research on RC frame structures reviewed the effectiveness of existing DBD procedures by undertaking a number of case study structures. One observation was that taller RC frame structures may often have a design displacement greater than the peak spectral displacement demand. This observation helped prompt the introduction of the general requirements on

K_{emax} discussed in the previous section. The refined recommendations for RC frame structures finally included in the model code were verified through consideration of a suite of case-study structures of 6, 9 and 12 storeys in height. One particular aspect that was studied was the recommendation in the model code to neglect the gravity loads in setting the flexural strength of beams. For comparison purpose the seismic design of the frames with nine and twelve storeys was undertaken with and without distributed gravity loads considered. The application of the design procedure was shown to be simple and useful. The actual behaviour of designed frames was investigated by performing pushover and non-linear dynamic analyses of the case study structures. A group of spectrum-compatible earthquake records characterized by increasing spectral displacement up to a period equal to 4 second were used. The envelope of displacement obtained from non-linear dynamic analyses was seen to be affected by the criteria adopted for distributing reinforcements in the columns. It was shown that distributed gravity loads on beams do not significantly influence the deformations recorded in the non-linear dynamic analyses, but their consideration in seismic design did considerably modify the forces that develop. The use of increased material strength and the non consideration of distributed gravity loads in the seismic design of beams helped limit the base shears to obtain a force-displacement response similar to that assumed in the design procedure. Furthermore, little differences were found between top displacement estimates from analyses and design. These results (refer Benedetti et al. 2008) indicate that use of the displacement-based design procedure enabled good control of the inelastic response, enabling performance-based design solutions to be realised.

Work on case-study RC wall and RC frame-wall structures provided support for the aforementioned capacity design recommendations included in the final chapter of the model code. Furthermore, various case studies reported by Rizzato et al. (2009), demonstrated that displacements could be better controlled using the displacement-based approach in comparison to traditional force-based design methods.

The research unit examining pre-cast RC structures also made some interesting observations about the relative performance of force-based and displacement-based design methods. Acknowledging that force-based design methods should not use the displacement ductility capacity to set a force-reduction factors (refer Section 2) the traditional FBD approach was modified by setting reduction-factors in proportion to the expected ductility demand. This improved FBD approach was then used to design a series of SDOF pre-cast column systems with the design objective of limiting storey drifts to 2.5%. The same SDOF systems were then designed using the DDBD approach, and non-linear time-history analyses were then undertaken using spectrum-compatible accelerograms to evaluate the performance of the different design solutions. Figure 3 compares the average of the maximum displacements for the force-based and displacement-based design solutions as predicted through non-linear time-history analyses.

The comparison made in Figure 3 is particularly interesting because it shows that while the DDBD solution has been fairly conservative, the level of protection against unwanted damage is fairly constant. This is in stark contrast to the FBD solutions in which it is seen that the potential damage appears to vary greatly as a function of the pier diameter selected. The variation in the FBD results has been attributed to the poor selection of cracked concrete section properties which codes recommend be set as a fraction of the gross section stiffness, without consideration of the strength assigned to the section. As such, this study by the pre-cast structures group illustrated how even when the ductility demand is used instead of the ductility capacity to set force-reduction factors, the design solutions can still be unsatisfactory and DDBD performance is much more consistent thereby enabling the provision of uniform risk design solutions.

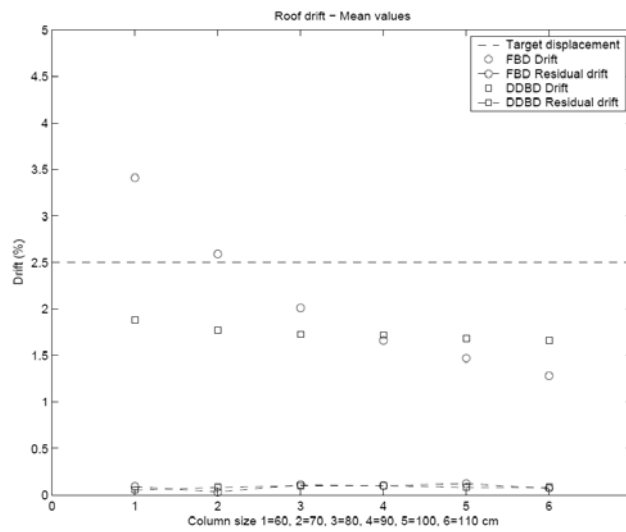


Figure 3. Comparison between target and recorded drift for pre-cast concrete columns of varying diameter, designed using both a force-based and displacement-based approach for a target drift of 2.5% and subject to spectrum-compatible non-linear time-history analyses.

Another useful set of guidelines developed for pre-cast concrete structures was to consider different base connection details in the seismic design of pre-cast structures. Four different connection types were studied and by considering how connection details influenced the hysteretic response, equivalent viscous damping expressions were calibrated for each type. As a result of this work, the model code includes the Eq.(5) for the equivalent viscous damping of pre-cast connections that are formed with grouted sleeves.

$$\xi_{eq} = 0.05 + 0.35 \left(1 - \frac{1}{\mu^{0.226}} \right) \left(1 + \frac{1}{(T_{eff} + 0.719)^{0.681}} \right) \quad (5)$$

Where μ is the displacement ductility demand and T_{eff} is the required effective period. Refer to the work by Belleri and Riva (2008) for more detailed information on the influence of pre-cast column base connections within a DBD context.

4.3 Steel Structures

The research into the displacement-based design of steel structures has focused on three types of steel structures: (i) steel moment resisting frame structures, (ii) steel frames incorporating buckling-restrained braces, and (iii) concentrically braced steel frames that utilise inverted V-bracing. Investigations into all three systems proceeded by forming trial methodologies (which required estimates of equivalent viscous damping and displaced shape) and evaluation of the performance of the design methods through advanced non-linear dynamic analyses. The new model code includes design expressions for the various steel systems. However, arguably the most significant findings have been obtained for the concentrically braced frame systems. The main feature of inverted V-braces is that the beam flexural stiffness must be explicitly taken into account in the design method, because it strongly affects the system response. This poses an additional difficulty for the analytical derivation of the displaced shape of V-braces. In fact, in addition to the cross section area and moment of inertia of braces and columns, the beam cross section second moment of area is also required. This aspect, which is specific of V-braces, is additional to other issues that characterize braced

structures in general, such as (i) the small redundancy, which makes the system prone to weak storey mechanisms, (ii) the strong redistribution of column axial strain from the elastic through the inelastic range of response.

As a consequence of the previous issues, it emerged from both the theoretical and numerical studies that controlling the inelastic displaced shape of braced structures requires a careful selection of elastic member properties (refer Della Corte 2006). In other words, controlling the inelastic displaced shape requires controlling also the elastic vibration mode, because a strict relationship exists between the initial response in the elastic field and the subsequent plastic behaviour of the structure.

Equations have been proposed for calculating both the elastic and inelastic vibration modes. The proposed approach in the model code is based on the kinematics of braced structures, where the flexural and shear deformability of each storey are summed. The strict relationship between the response at buckling of the braces (which signals the end of the elastic system response) and the fully inelastic displacement response emerges clearly from the Equations.

In particular, in the proposed method, for a given distribution of brace and column slenderness, the required cross-section area of braces and columns, and the flexural stiffness of beams, can be calculated once the desired collapse mechanism has been fixed.

Numerical studies have been carried out in order to verify the efficiency of the proposed design Equations. All the numerical tests suggest that the procedure works properly and the response of structures designed according to the DDBD methodology is better than the response of similar structures designed according to standard and codified force-based design procedures. Figure 4 shows the results of a verification of a 10-storey concentrically braced frame structure designed using a simplified version of the developed DDBD method.

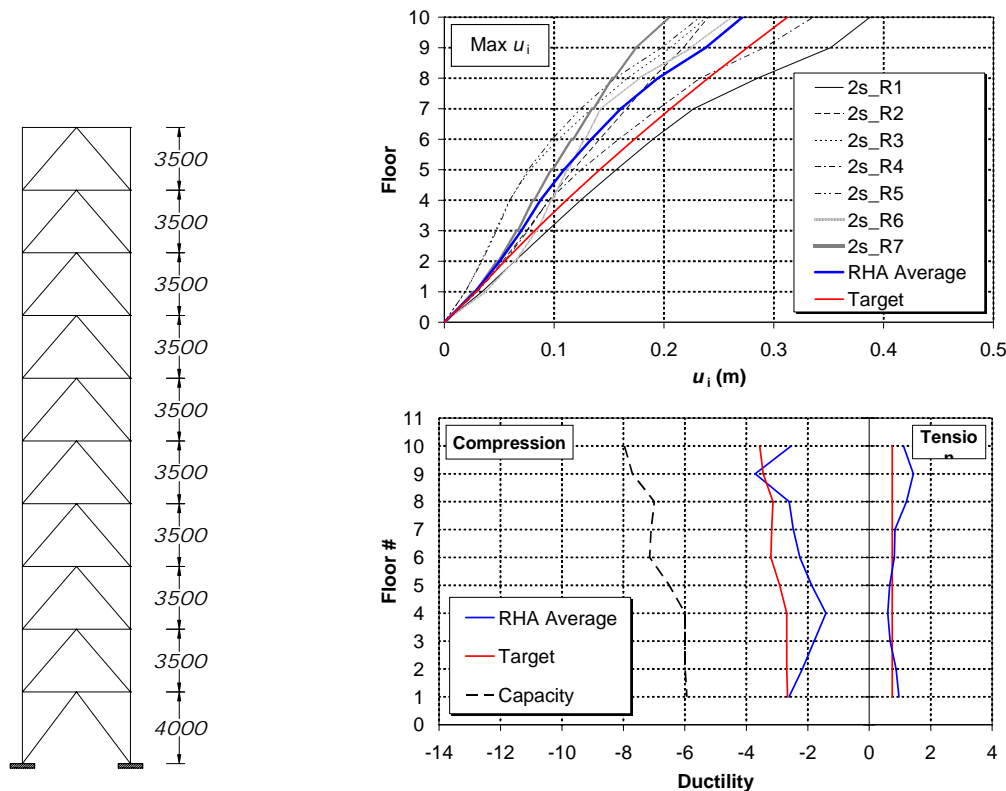


Figure 4. Results of case study verification analyses of a 10-storey concentrically-braced steel frame designed using DDBD (i) elevation of case study structure, (ii) maximum displacements compared to design profile, and (iii) average ductility demands compared to target values.

It is worth emphasizing that the numerical studies have also shown that some degree of empirical judgment must be placed on the selection of appropriate distributions for brace and column slenderness, which is an input choice in the design process. In particular, it was found that inappropriate distributions of brace slenderness may lead to design solutions that are only theoretical, because of the impossibility to find commercial shapes having the properties coming out of the design process. As such, some trial and error could be required before reaching a satisfying and feasible design solution. For more detailed information on the proposed DBD procedure for CBF systems refer to Della Corte and Mazzolani (2008).

It was noted for some case study structures that dynamic effects could be significant and the variability in the response may be sensitive to the model of elastic viscous damping selected for the non-linear time-history analyses. This was identified as an area requiring further research. In addition, the equivalent viscous damping expression utilised for the design of CBF systems was taken as the Takeda model. In reality the hysteretic response of braced frames can be quite different from the Takeda model and separate research by () has recently developed an expression for the equivalent viscous damping of CBF systems. Future research into the DDBD of steel structures should look to develop guidelines for eccentrically braced frame systems, as well as dual systems possessing steel moment-resisting frames and bracing systems in parallel.

4.4 Unreinforced Masonry Structures

Given the relatively limited work that has been done previously into the displacement-based design and assessment of masonry structures, a large amount of work is required to form of a comprehensive set of displacement-based design and assessment guidelines for masonry structures. In this project a DBD methodology was outlined for masonry structures, but in order to test this a number of modelling developments were required. As such, a significant amount of work focussed on the mechanical response of masonry piers and on the dynamic non-linear response of simple masonry structures. It was found that modern earthquakes have shown that masonry buildings often collapse due to the formation of soft-storey mechanisms, which suggests the structures can be considered as SDOF systems. This should enable simplifications in the design or eventual displacement-based assessment procedures.

In terms of modelling developments, work has been done to consider the difference between the peak load and the residual strength on the overall behaviour of a masonry structure. A constitutive model was developed to enable detailed analyses and to estimate the hysteretic damping of the structures, that maintains the simplicity of an elastic-perfectly-plastic model, but that takes account of the energy absorbed by the cohesive forces acting within the walls. The research unit has proposed the use of two different equivalent viscous damping expressions for masonry structures, one for masonry systems that develop a shear sliding mechanism and the other for masonry systems that develop a diagonal cracking mechanism. The expressions were obtained considering the results of a limited number of experimental studies and further research is required to fully develop a set of guidelines for the displacement-based design and assessment of masonry structures.

4.5 Timber Structures

For the development of DBD guidelines for timber structures, the focus was on warehouses or commercial building structures formed glue-laminated timber portal frames as these are commonly built in Italy. In addition, the work focussed on the response of dowelled connections, as these were considered to represent the type of connection most extensively used in glulam construction technology in Italy.

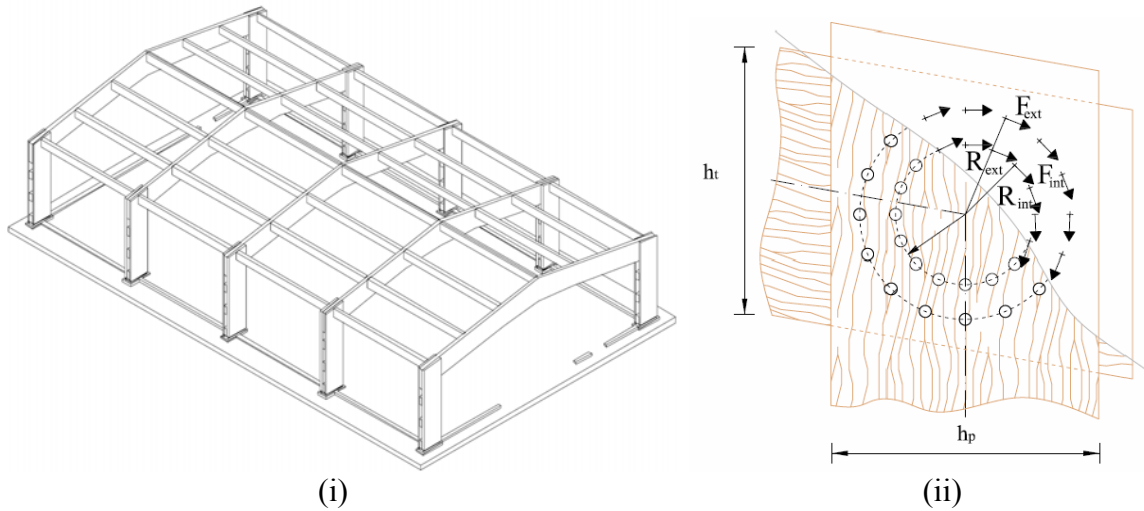


Figure 5. Illustration of the timber portal case study structures of Zonta et al. (2008) (i) perspective view of portal structure, and (ii) annular bolted moment resisting connection arrangement.

In the model code strain limits have been set for timber to ensure that the timber does not yield. Inelastic deformations in timber structures should occur in ductile connections. For timber framed wall structures this can occur in the nailed connections and drift limits were set in line with the observations made by Folz and Filiatrualt (2001) and Filiatrualt and Folz (2002). It is noted that the experimental work undertaken on timber framed structures in support of the DDBD recommendations is fairly limited and future work in this area may lead to changes in the model code recommendations. For the timber portal structures it was recognised that it would be advantageous if the target displacement Δ_d of a portal could be estimated a priori, and the relation between equivalent damping ratio ξ_d and ductility μ , without prior knowledge of the final section dimensions or connection strengths. The formulation of target displacement Δ_d has been first theoretically stated and later refined, based on a calibration carried out using a Monte Carlo simulation. A total of one thousand different case studies were generated, assuming geometrical dimensions, joint configuration, and mechanical properties of materials as random variables. A statistical procedure was developed to select the geometrical dimensions of the members simulating the logical path commonly followed by the designer. In a similar way, a distribution of the mechanical properties were considered in order to reproduce the actual variability of the materials (glulam and steel) available in the market. For each sample, the actual ultimate displacement of the structure was calculated using non linear static pushover analysis. This analysis resulted in the following practical expression for the target displacement for the damage control limit state:

$$\Delta_d = c_1 \cdot \delta_{u,k} \cdot \frac{\theta \cdot \gamma \cdot \beta}{1 + 1/\theta \cdot \gamma \cdot \beta} \cdot \left(1 - \frac{q_{GRAV}}{q_{LIM}} \right) + c_3 \cdot H \cdot (\theta + 1) \cdot \gamma \quad (6)$$

where: $\delta_{u,k}$ it is the characteristic ultimate sliding capacity of the dowel; θ is the aspect ratio of the portal, $\gamma=L/h$ is the rate between the span of the portal L and the height of the members cross-section h ; β is the rate between h and the external radius of the moment resisting joint; q_{GRAV}/q_{LIM} is the rate between design and ultimate gravitational loads; H is the height of the portal; $c_1 = 1.15$ and $c_3 = 0.00555$ are constants resulting from the calibration. As for the equivalent viscous damping ξ_d , the classical expression typically used was selected:

$$\xi_d = \xi_0 + \frac{a}{\pi} \left(1 - \frac{1}{\mu^{0.5}} \right) \quad (7)$$

and values were set for parameter a and ξ_0 based on the experimental outcomes of tests on full-scale fastened glulam joints, carried out according to UNI EN 12512. This load protocol includes a sequence of reversed cycles of increasing amplitude and is the standard used in Europe to characterize the response of fastened joints. The general assumption is that, for fastened connections, the hysteretic dissipation is mostly due to the steel dowels that embed in the wood during the load action. Because this mechanism implies a reduction of energy dissipation after each cycle, the total amount of energy dissipated is strictly dependent on the load protocol. Taking account of this issue, three different types of damping-to-ductility curves were defined. The first curve is that obtained under the same load protocol used in the experimental tests, while the other two represent the upper- and lower-bond curves, theoretically expected in the case of monotonically increasing cyclic load and constant amplitude cyclic load, respectively. In order to validate the design procedure Zonta et al. (2008) applied the Direct DBD method to three representative case studies. Using pushover non-linear analyses, it was demonstrated that the expression provides lower bound values of the displacement capacity that are close to those obtained through back-analysis using a much more refined model. On the other hand, the simulations also illustrated that the design base shear is very sensitive to the dissipation capacity of the joints. This is an area for future research required into the capacity design of timber structures. The comparison with the results of the current Eurocode 8 procedure shows that the DBD method can potentially overcome some of the limitations of the traditional Force Based Design methods.

4.6 Bridges

Guidelines for the DDBD of bridge structures had already put forward in a number of publications including that of Priestley, Calvi and Kowalsky (2007). As such, the main activities of the research unit studying bridge structures has been to consider whether the existing guidelines could be extended to irregular bridge configurations as well as bridge systems with tall piers. In order to achieve this a large number of case studies of both regular and irregular RC bridges were undertaken in this research project so as to develop the existing displacement-based design guidelines. In nearly all case studies examined for irregular bridges, it was observed that the DBD procedure performed rather well, whereas the FBD approach had difficulty in predicting the displacement profile and moments for the deck. An example case study bridge arrangement is shown in Figure 6 and results of non-linear time-history analyses for the bridge for two different pier heights is shown in Figure 7.

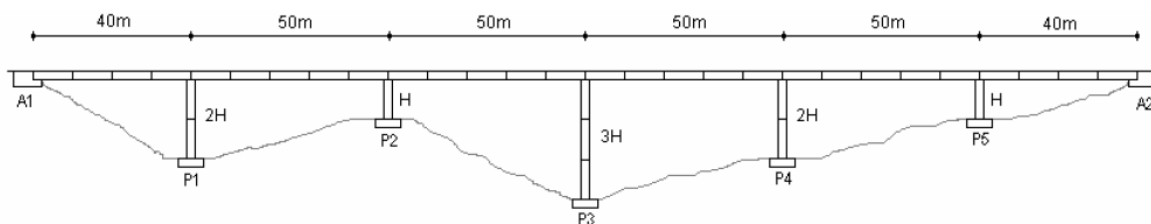


Figure 6. Example of a case study bridge configuration examined by the RC Bridge research unit.

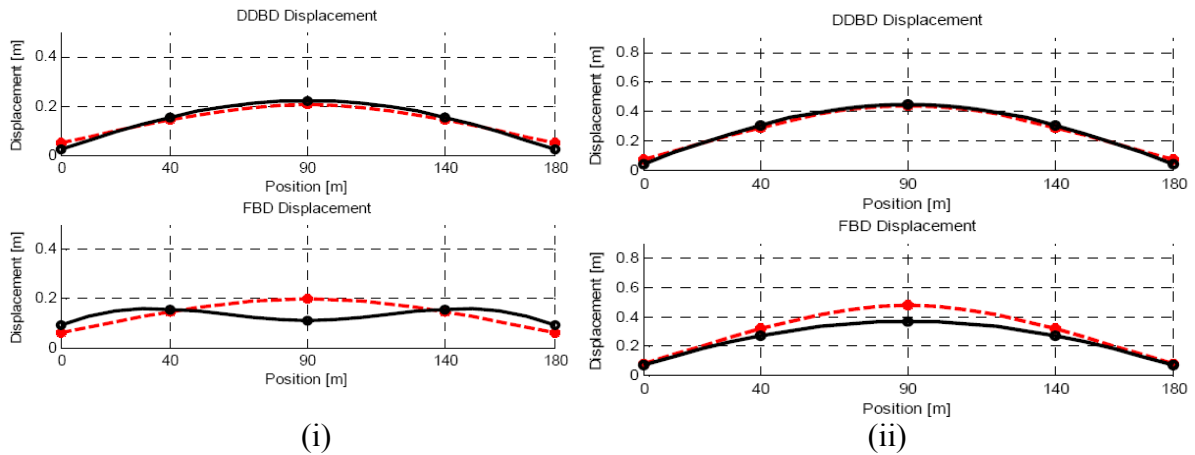


Figure 7. Comparison of DDBD (upper) and Force-Base Design (lower) predictions with non-linear time-history results for the transverse response of the case study bridge presented in Figure 6 with (i) $H = 7.5\text{m}$ and (ii) $H = 15\text{m}$.

The relative performance of the methods for the short and tall pier scenarios presented in Figure 7 is worth considering carefully. It is seen that for the same proportions of a structure the force-based design approach may underestimate or overestimate deformations, and therefore damage. This highlights what is considered one of the main shortcomings with force-based design; difficulty in providing uniform risk solutions. Note that the DDBD procedure performs excellently for both cases.

To further improve the DDBD approach and capacity design recommendations, the following design refinements and recommendations have been proposed through the course of the work:

- *Distribution of shear force for design.* In general, for bridges with regular span and moderate pier heights, axial load distributions at pier bases are almost the same. For ease of construction, piers also have same cross-sections and reinforcement content. Thus moment capacity of all the piers is the same. Consequently, the shear force is distributed in proportion to inverse of the pier height or to ratio of displacement ductility demand and pier height, if the pier is yielded or not. For bridges with significant irregularities in span and pier heights (as the ones studied here) the assumption of equal axial load for all the piers is not valid and hence moment capacities are different at different pier bases. In these cases we suggest to represent the moment capacities as a percentage of moment capacity of the critical pier.
- *Modelling approach for tall piers.* Piers are commonly modelled as a single element with a percentage of mass lumped at top: this is valid for moderate height piers. For tall massive piers, lumping a percentage of mass at the top was found to underestimate the base shear significantly. Thus, either a distributed mass model or discretized pier elements are required where masses can be lumped at the intermediate nodes. Since the distributed mass model is computationally expensive, a lumped mass model is recommended. A modified equation is proposed that uses basic principles of statics and considers the inertia forces as acting at intermediate nodes to calculate an equivalent secant stiffness of the piers. The secant stiffness is obtained from the equation.

$$E_c I_{\text{sec}} \Delta_{\text{top}} = \left(V_B - \sum_{i=1}^{N-1} F_i \right) \frac{H^3}{3} + \sum_{i=1}^{N-1} \frac{F_i h_i^2}{2} \left(H - \frac{h_i}{3} \right) \quad (8)$$

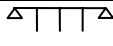
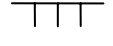
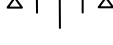
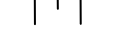
where, V_B and H are the base shear and overall height of the pier under consideration respectively, F_i 's and h_i 's are the inertia force and height of its application from base of the pier respectively. E_c is the modulus of the elasticity for concrete, I_{sec} is the secant moment of inertia of the pier section at base and Δ_{top} is the design displacement at pier top.

- For capacity design, a new modal superposition principle to consider higher mode effects at the potential flexural plastic hinge location for elastic and/or limited ductile pier could be used. In particular, effective modal superposition with a 5% damped acceleration spectrum is used to estimate the abutment shear and deck transverse moment, whereas effective modal superposition with the design displacement spectrum scaled down by an appropriate system damping factor is used to calculate the flexural moment demand at the potential plastic hinge locations (when higher modes are equally important as first inelastic mode).

Seven bridges were studied with different configurations to consider the effectiveness of these recommendations. All the bridges were designed according to refined DDBD methodology, compared with original DDBD and FBD procedures and validated by means of non-linear time history analyses. The results indicated that the above recommendations lead to better estimates of the inelastic displacement demand on the structures and lead to reinforcement quantities which are always more efficient than those obtained from FBD.

For the model code, an effort was made to simplify the DDBD procedure for the transverse direction. Design in the transverse direction might typically be considered difficult because the displaced shape is often uncertain and an iterative procedure is therefore required in order to establish an appropriate displaced shape. To simplify the procedure in the model code, Table 8 has been included together with Eq.(9) to Eq.(11).

Table 8. Displaced shape profiles provided for design of bridges in the transverse direction.

Pier Configuration		Abutment Type	Displacement Shape
Uniform		Pinned	Eq. 9
Uniform		Free	Eq. 10
Valley		Pinned	Eq. 9
Valley Ridge		Free	Eq. 11

$$\delta_i = \frac{16}{5L^4} (x^4 - 2Lx^3 + L^3x) \quad (9)$$

$$\frac{\delta_i}{\delta_c} = 1.0 \quad (10)$$

$$\delta_i = 1.5 - \frac{8}{5L^4} (x^4 - 2Lx^3 + L^3x) \quad (11)$$

In addition to the displaced shape, some account of higher mode effects has been incorporated through the specification of a higher mode deformation factor. This factor effectively reduces

the design displacement to account for the additional deformations that will be caused by higher modes and which are not directly controlled through the displacement-based design procedure. A general procedure for estimating the higher mode deformation factor is provided. However, it is expected that future research will better quantify the magnitude of higher mode deformations and that the factor will then be updated.

4.7 Composite Structures

Only limited developments were made during the course of the research for composite structures. Work was done to consider the best modelling possibilities for composite systems and various case study structures were investigated. However, further research is required to clearly identify deformation limits for different performance levels in composite structures, to identify likely displacement profiles at peak response, and to develop equivalent viscous damping expressions that consider the likely hysteretic characteristics of composite systems.

4.8 Isolated Structures and Structures with Added Damping

Two Direct Displacement-Based Design (DDBD) procedures for buildings and bridges with seismic Isolation Systems (IS's) have been developed, implemented and tested within the project. The proposed design procedures have been realized considering different idealized force-displacement cyclic behaviours, which may be used to describe the response of a wide variety of IS's, including: Lead-Rubber Bearings (LRB), High-Damping Rubber Bearings (HDRB), Friction Pendulum Bearings (FPB) and combinations of Flat Sliding Bearings (FSB) with different auxiliary devices (e.g. SMA-based re-centring devices, Viscous Dampers (VD), Steel energy dissipating components, etc.).

The design philosophy of the proposed procedures is based on the general requirement that the full serviceability of the structure should be maintained after the design seismic event, in order to avoid any repair or interruption of activities. The key aspect of the proposed design procedures is the target displacement profile of the structure. It is specified by assigning a predetermined displacement pattern and a target displacement amplitude to the structure.

For Base Isolated (BI)-buildings, the displacement pattern is derived from an approximate expression of the first modal shape of the BI-building. The target displacement amplitude is then assigned, by selecting suitable values of the maximum IS displacement and maximum storey drift, which comply with a full serviceability performance level of the building.

For bridges with IS, a rigid translation of the deck in the transverse direction is assumed, for both continuous and simply supported deck bridges. The target displacement amplitude of the deck, in both the longitudinal and transverse direction, is assigned to comply with a full serviceability performance level of the bridge, expressed through a limit value of the maximum displacements of IS's, piers and joints.

Several design examples and validation studies through Nonlinear Time-History Analyses (NTHA) have been performed to verify the accuracy of the proposed design procedures. A set of seven natural and artificial ground acceleration-time histories, compatible (on average) with a DBD-adapted version of the displacement response spectrum provided by the Eurocode 8 for soil type C, has been used in the numerical simulation analyses.

Several configurations of framed buildings, differing in (i) number of storeys (3, 5 and 7), (ii) IS types (HDRB, LRB, FPB with or w/o VD and FSB+SMA with or w/o VD) and (iii) target IS displacements and maximum storey drifts, have been examined within comprehensive parametric analyses. The results presented in Table 9 (Cardone et al. 2008) and in particular the ratio of the maximum storey drift to the design storey drift ($\theta_{SAP}/\theta_{Exp}$), illustrate that the method controls the displacement response excellently for a wide range of isolation systems.

Table 9. Results of verification analyses (from Cardone et al. 2008) of 3, 5 and 7-storey buildings designed with different base isolation systems (IS).

Conf.	IS	T_{fb} (sec)	T_{IS} (sec)	ξ_{IS} (%)	D_{d-Exp} (mm)	θ_{Exp} (%)	D_{d-SAP} (mm)	θ_{SAP} (%)	F_{d-Exp} (kN)	F_{d-SAP} (kN)	$D_{d-SAP}/$ D_{d-Exp}	$\theta_{SAP}/$ θ_{Exp}	$F_{d-SAP}/$ F_{d-Exp}
3 Piani	HDRB	0.37	1.32	21.6	150	0.25	137	0.242	1358	1477	0.91	0.97	1.09
	LRB								133	0.215	1358	1262	0.89
	FPS										1.01	0.91	1.00
	SMA	3.19	23.6	350	0.10	356	0.091	543	545	1.02	0.91	1.00	
5 Piani	HDRB	0.57	2.10	19.4	250	0.25	246	0.281	1342	1549	0.98	1.12	1.15
	LRB								274	0.267	1342	1436	1.10
	FPS										0.98	0.94	0.99
	SMA	2.85	18.3	350	0.20	332	0.185	1020	995	0.95	0.93	0.98	
7 Piani	HDRB	0.78	2.92	19.3	350	0.25	343	0.276	1295	1469	0.98	1.10	1.13
	LRB										378	0.266	1295
	FPS										1.00	1.00	1.00
	SMA										0.92	0.97	0.96

As further verification, the proposed design procedure has been applied to a case study, given by a 4-storey, 2-bay, three-dimensional RC frame designed for gravity loads only. The comparison between DDBD predictions and NTHA results confirmed the accuracy of the design procedure in the attainment of the target displacement profile of the BI-building, with percent differences lower than 10-15% in terms of IS displacements and lower than 15-20% in terms of maximum interstorey drifts, regardless the IS type used. The NTHA results, however, also pointed out that, in order to get an accurate estimation of the stresses in the structural members, the lateral force distributions to be used in the Linear Static Analysis (LSA) of the building must reflect the actual mechanical behavior of the IS. A distribution of lateral forces proportional to the storey masses (like that suggested in the standard formulation of the DDBD method) may be suitable for (quasi-)elastic isolation systems (e.g. LDRB, HDRB and LRB with low degree of non linearity) while do not for strongly nonlinear IS's (e.g. FPB, FSB+SMA and LRB with high degree of non linearity).

An extensive parametric investigation of NTHA has been carried out with the aim of defining accurate distributions of equivalent static forces, specific for each IS, that can be used to ensure successful capacity design. A new approach for the evaluation of accurate lateral force distributions for the LSA of BI-Buildings has then been proposed. The proposed enhanced lateral force distributions are proportional to the displacement profile of the structure, which is expressed as a linear combination of the first three approximate modal shapes of the BI-building, with the IS modelled through its effective stiffness at the design displacement. For this reason, the acronym 3-MM (3-Mode Method) has been used to identify the proposed method. The combination coefficients of the modal shapes are expressed as a function of a number of IS parameters, accounting for its actual nonlinear mechanical behaviour and isolation ratio of the BI-building. The guidelines for this method are included within Annex 4 of the new model code.

Further studies, based on NTHA of SDOF models of BI-buildings, have been conducted to assess the accuracy of the Jacobsen's equivalent damping approach (Jacobsen, 1960), used in the DDBD (along with the secant stiffness method) for the linearization of the structure. The

results of these studies essentially confirmed the suitability of the adopted linearization approach in the evaluation of the maximum displacements of buildings equipped with different IS types.

The DDBD procedure for bridges designed with Isolation Systems (IS) has been verified through extensive NTHA, carried out on different configurations of multi-span continuous and simply supported deck bridges, differing in the layout of pier heights, IS types and associated mechanical parameters. Further NTHA have been conducted on a case study given by a 10-span simply supported deck bridge of the A16 (Napoli-Canosa) Italian Highway. The comparison between DDBD predictions and NTHA results effectively confirmed the reliability of the proposed design procedure in the attainment of the target displacement profile of the bridge, which is characterised by a uniform translation of the deck in the transverse direction. The attainment of the target displacement profile implies the achievement of the performance objectives of the design, which can be summarised as follows: (i) elastic behaviour of the piers, (ii) maximum IS displacements lower than their ultimate displacement capacity and (iii) adequate margin with respect to the gap closure of the joints. Obviously, a suitable compromise between the optimal IS configuration provided by the design procedure and a number of economic/practical needs must be pursued. Basically, this means to limit the number of different types of IS devices to one or two, thus accepting some errors in the attainment of the target displacement profile of the bridge.

Overall, significant developments have been made for the seismic design of isolation systems in this work and the model code incorporates many of the new findings. Future work in this area could aim to simplify some of the procedures and consider the development of design displacement profiles for different forms of building structure.

4.9 Foundation Structures

The main, and ambitious, goal for research into the DDBD of foundation structures in this project has been to define a suitable procedure for seismic design of foundations in the framework of Direct Displacement Based Design (DDBD), accounting for non-linear soil-foundation interaction. The recent development of performance-based approaches for seismic design has raised awareness of the potentially relevant role of nonlinear dynamic soil-structure interaction (DSSI) effects on the response of the superstructure but that on the other hand there is currently a lack of suitable methods and experimental results in the literature required to support any design approach based on nonlinear DSSI concepts.

The lack of existing consolidated approaches has led the research into the DDBD of foundation structures to focus, in the first half of the project, on the recovery of experimental data and development and calibration of numerical, albeit simplified, methods. The second half of the project was devoted to devise a suitable procedure to account for nonlinear DSSI in the framework of DDBD. Due to the innovative character of this research, priority was given to the problems related to the development of the procedure itself, rather than on the complexity of the structural types to be investigated. Therefore the research was mostly limited to shallow foundations of bridge piers, although, with suitable modifications, it could be extended to other structural types and deep foundations.

A synthesis of our research activities and findings related to foundation structures is as follows.

(a) Recovery of existing experimental data on the seismic behaviour of shallow foundations, making essentially reference to the results from the large-scale cyclic tests carried out at the JRC in Ispra during the European TRISEE project (1996-98), and, some years later, at the PWRI in Tsukuba, Japan, in the framework of the research cooperation agreement between the Public Works Research Institute and the Politecnico di Milano. Mostly important to shed

light on the role of nonlinear DSSI were the shaking table tests carried out at PWRI within the same agreement.

(b) Calibration, based on the previous experimental results, of nonlinear constitutive models at different level of complexity, but with the common idea to represent the soil-foundation system by a single macro-element. This approach allowed fast and reliable parametric analyses to be undertaken, without making use of more sophisticated but time-consuming and difficult-to-calibrate finite elements approaches. The research activity on the numerical simulation of the PWRI shaking table tests has lead to the publication of Paolucci, Shirato and Yilmaz (2008).

(c) Based on the parametric analyses carried out within activity (b), several graphs and simplified formulas were proposed, suitable to capture the foundation stiffness decay and the corresponding increase of damping of the soil-foundation system, as a function of rocking and displacement of the foundation itself. Curves such as those shown in Figure 8 below (from Paolucci et al. 2009) are included in Annex 3 of the model code to aid designers consider foundations within the DDBD procedure.

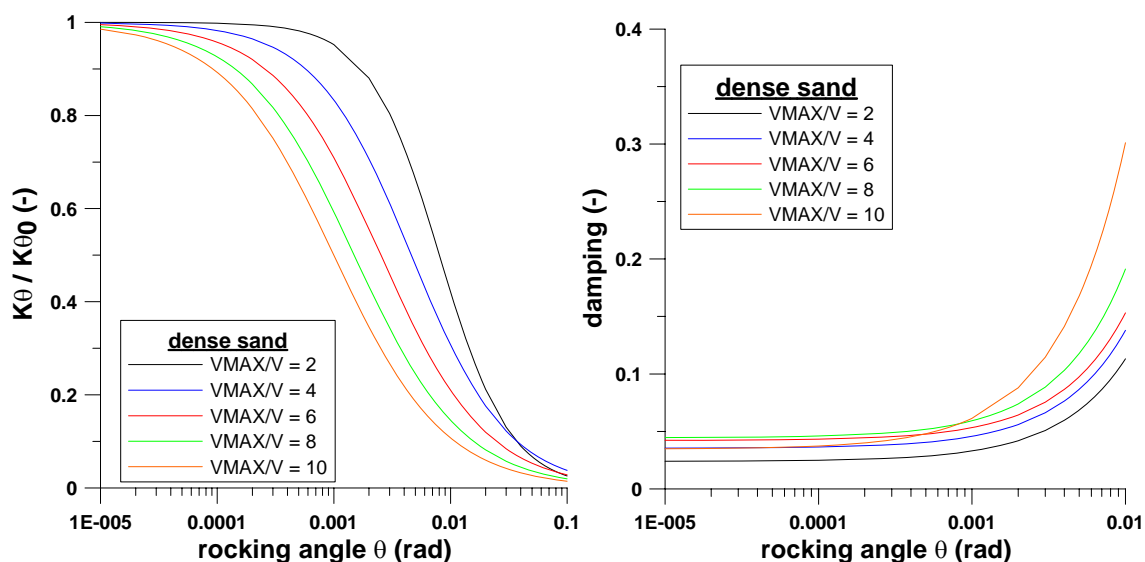


Figure 8. Variation of rotational stiffness (left) and equivalent viscous damping (right) for a shallow foundation on dense sand (from Paolucci et al. 2009).

(d) Development of a numerical code in Matlab to study the dynamic interaction between a single dof nonlinear oscillator with a foundation modelled by a nonlinear macro-element. The interesting feature of this software is to model the nonlinear DSSI in a simplified albeit rationale and physically sound way, with the purpose to analyze in which conditions the nonlinear DSSI effects may play a role in the seismic response of the super-structure.

(e) Finally, a procedure to account for non-linear DSSI in the DDBD was devised, in cooperation with the bridges research unit, making special reference to bridge piers on shallow foundation, with the production of different examples of application. A synthesis is presented in publication Paolucci et al.(2008).

Clearly, there has been a considerable amount of valuable work done into the seismic design of foundation structures and a number of useful publications have been produced. However, there is still a large amount of research required in this field as, for example, viscous damping for deep foundations is still uncertain and foundations on cohesive soils have not yet been studied. This should be the attention of future research projects into DDBD.

4.10 Retaining Structures

Very little work had been done into the displacement-based design of retaining structures prior to this research project. The first challenge for the research unit examining retaining structures was therefore to consider how a DDBD procedure could be formulated for such systems. The work has focussed on the development of a DDBD approach for cantilever diaphragm wall structures embedded in coarse-grained soils, with and without anchors. Through collaboration with research unit 1, it was recognised that the soil-structure system could be represented as an equivalent SDOF system by considering layers of soil that would displace during a seismic event, as illustrated in Figure 9.

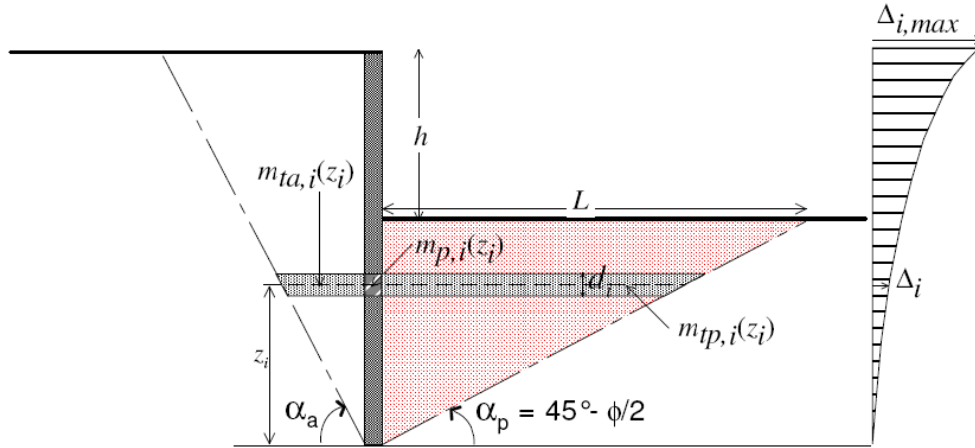


Figure 9. Discetisation approach used in the equivalent SDOF representation of retaining structures.

Some consideration has been given to the deformation limits to be used as part of a DBD approach. In collaboration with research unit 11, it was agreed that soil strains should be limited in relation to the effective reduction in the soil modulus (see Table 6). However, it was also considered that the major performance criteria for retaining structures that do not form part of the foundations of buildings, is the residual deformation that remains in the walls following an earthquake event. Limits on residual deformations for retaining structures have been specified in the model code as shown in Table 7. Note that there is not a strong scientific background to these residual deformation limits and future research could prompt changes. Clearly, the peak displacement is likely to be larger than the residual displacement. However, as little is known about the likely relation of peak to residual deformations of retaining structures, it has been proposed in the model code that the residual deformation be assumed equal to the residual deformation. By assuming a displacement profile for the wall, the equivalent SDOF characteristics of the MDOF soil-structure system could then be determined and a standard DDBD approach followed (see Cecconi et al. 2006).

Another significant proposal for the retaining walls DBD procedure has been to propose an equivalent viscous damping value (ξ_e) for the system in accordance with Eq.(12).

$$\xi_e = \frac{\xi_{ta} \sum_{i=1}^n m_{ta,i} + \xi_{tp} \sum_{i=1}^n m_{tp,i} + \xi_p \sum_{i=1}^n m_{p,i}}{\sum_{i=1}^n (m_{ta,i} + m_{tp,i} + m_{p,i})} \quad (11)$$

Within this evaluation the soil damping (active, ξ_{ta} , and passive, ξ_{tp}) components and the wall damping component, ξ_p , have been factored by the relative masses. It has been proposed in

the model code that the damping components of the soil can be related to the soil shear strain in line with the work of Vucetic and Dobry (1991). While the model code has included these recommendations to enable designers to familiarise themselves with the DDBD approach, it is considered that additional verification analyses may be required to fully verify the equivalent viscous damping of the complex systems.

Different analysis procedures available in the current engineering practice were critically reviewed by the retaining structures research unit to consider how DDBD solutions compare with more traditional methods. Comparison of the design base shear required by the DBD calculation and more traditional pseudo-static design approaches (e.g.: Mononobe-Okabe method) suggested that the former approach is likely to be more realistic than the current pseudo-static analysis, despite its immediateness and simplicity. Numerical analyses were performed using the commercial finite-element-method (FEM) code PLAXIS.v.8.5. The comparison between the results of the method and those obtained from dynamic numerical analyses has shown fair agreement. However, it was noted that an adequate choice of the soil masses participating in the seismic event - i.e. the geometry of the active/passive soil wedges - and of the equivalent damping are fundamental in the calibration of the design procedure. To this aim, the availability of well-documented case histories is strongly needed and should constitute one of the main objectives of future research. Clearly, there is also a large amount of additional research that could be undertaken in order to consider other forms of retaining structures and other considerations such as sloping backfills, water table effects, and cohesive soils.

5 DISCUSSION

Overall this project has been very successful in achieving a set of ambitious objectives. The main product has been the formation of a model code that may be considered a world first. This was only possible because of excellent leadership and active collaboration between internal research units. In addition to the model code, many high quality publications and developments to the DDBD procedure have been made. However, not only has this research been successful on a technical front, but it has also been a very effective means of increasing awareness of the capability of displacement-based design methods throughout Italy. With greater numbers aware of the DDBD approach it is expected that further developments to the guidelines and state-of-the-art will occur more rapidly.

In terms of management considerations, it was noted that collaboration between universities was assisted by having a fairly limited number of research units. However, collaboration tended to occur between research units that were located geographically close to each other. This could be considered for future research proposals in which collaboration might be desired.

It should be noted that progress in some cases was significant, as seen in previous section. However, in other cases only limited progress was achieved during the three year research project. Two examples would be the cases of composite structures and masonry structures. As there was very little existing literature on the DDBD of such structures it appears that the research line goals of developing a set of guidelines for such systems were possibly too ambitious. However, there is also a management challenge in such situations, to encourage research units to seek assistance in overcoming some of the significant technical challenges they may encounter. Clearly, because of the limited progress that was made, DDBD of composite and masonry structures will need to be the subject of future research projects. There are of course a number of other research areas that also need development. In

particular, while an excellent start has been made, further research into foundation structures and retaining structures would be very beneficial.

A final point for discussion is the difficulty that researchers face when trying to compare DBD and FBD procedures. The code-based FBD approaches require a number of decisions to be made in relation either to the structural modelling or choice of design parameters, such as the structural behaviour or reduction factor. Without clear guidance as to the exact modelling and design approach, two different designers may obtain very different solutions using the same code procedures. As such, the comparison between DBD and FBD procedures can usually only be indicative.

6 VISIONS AND DEVELOPMENTS

The work undertaken by research line IV of the RELUIS project has seen the development of a world first for earthquake engineering; a model code for displacement-based seismic design. The vision is that this document will be considered and refined as research in the field continues in years to come, and that it will be eventually developed into a formal code for displacement-based seismic design.

The code is the main product of a three year research project involving 11 different universities throughout Italy. The work undertaken by the different research units has clearly demonstrated some of the benefits and advantages of displacement-based seismic design over current code force-based design methods. Some of the findings and recommendations made in the work are preliminary and future research will endeavour to validate or refine the current provisions in the model code. However, the production of the model code and the set of supporting case study examples will mean that in the future the recommendations for DDBD can be more easily developed, refined and extended to an even greater number of structural typologies.

Another area that was identified as a possible area for research at the start of the RELUIS project was the displacement-based assessment of structures. Limited progress was made towards this goal principally because of the large number of uncertainties that needed to be overcome to firstly enable displacement-based design guidelines to be formed. However, considering the seismic risk associated with the existing building stock in Italy, it is clear that the possibility of providing practitioners with more effective and more reliable tools for seismic assessment is very attractive. As such, in addition to refining the design guidelines, the next major goal for research in this field may be the formation of model code for the displacement-based seismic assessment of structures.

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