DEVELOPING GUIDELINES FOR DISPLACEMENT-BASED SEISMIC ASSESSMENT

Gian Michele Calvi \textsuperscript{a}, Timothy John Sullivan \textsuperscript{b}

\textsuperscript{a} IUSS Pavia, Pavia, Italy, gm.calvi@iusspavia.it
\textsuperscript{b} University of Pavia, Pavia, Italy, timothy.sullivan@unipv.it

1 INTRODUCTION

The need for accurate seismic assessment methods is particularly evident in Italy, a country that has historically suffered greatly from earthquakes. Recent seismic events such as the 2009 L’Aquila and the 2012 Emilia-Romagna earthquakes have increased public awareness of the risks posed by earthquakes, but there is concern that these events may be forgotten without changing attitudes and practices for seismic assessment and retrofit. Furthermore, these recent earthquakes are relatively small events for Italy if one considers the estimated 150,000 people killed in the Val di Noto earthquake of 1693 or the 50,000 people who lost their lives in the Calabria earthquake of 1783.

One might argue that modern engineering should certainly have reduced the risk of earthquakes with respect to the 17th and 18th centuries. However, in addition to historical buildings, a large proportion of the building stock in Italy was realized in the 1950’s and 1960’s during a period of rapid urbanization with few controls in place to ensure good quality construction and design solutions developed using building codes with very minimal seismic considerations.

Traditional seismic assessment methods have tended to rely on a simple comparison of estimated base shear capacity and base shear demand specified by a code (Priestley et al. 2007). The required code base shear is found by reducing the elastic base shear force corresponding to the elastic stiffness of the structure, by a code-specified force-reduction or behaviour factor. The actual assessed base shear strength is then estimated and compared with the demand to identify whether the structure has enough strength to survive the earthquake. As pointed out by Priestley et al. (2007), the problems with this approach are that: (i) no assessment is made of the actual displacement or ductility capacity, (ii) no capacity design check is included to determine undesirable failure modes, and (iii) no estimate is made of the risk of a structure which is deemed to fail the strength check.

In recognition of the limitations of force-based design and assessment methods, the Direct Displacement-Based Design (DBD) approach of Priestley et al. (2007) has been proposed. This is just one of many different displacement-based methodologies (see Sullivan et al. 2003) but Direct DBD is the most developed displacement-based procedure and recently a Model Code for the Displacement-Based Seismic Design of Structures (Calvi & Sullivan 2009, Sullivan et al. 2012) was published as part of the work by research line IV in the 2005-2008 RELUIS project. The research undertaken in the 2005-2008 RELUIS project highlighted that Direct DBD can be used to provide effective seismic design solutions through relatively simple calculations. In the 2010-2013 project, the focus shifted towards development of the assessment procedure and this paper reviews the principal developments made.
2 BACKGROUND AND MOTIVATION

The text by Priestley et al. (2007) presents the fundamentals for the Direct DBA approach, which is reviewed here with reference to Figure 1 (from Sullivan and Calvi, 2013). As other assessment procedures, the first step (Figure 1a) requires examination of the structure to identify material properties, member sizes and the general geometry. The Direct DBA approach utilises the substitute structure concept of Gulkan and Sozen (1974) and Shibata and Sozen (1976) to represent the structure as an equivalent Single-Degree-Of-Freedom (SDOF) system (Figure 1b), characterised by a secant stiffness, $K_e$, at the displacement capacity $\Delta_{\text{cap}}$ (or any other limit state of interest) as shown in Figure 1c. To do this, the designer must first assess (by comparing the relative strengths of members) the inelastic mechanism that is likely to develop. For example, in the case of a RC frame structure either a column-sway (such as that shown in Figure 1b) or a beam sway mechanism might develop, but the assessment could also need to identify how the effects of smooth reinforcement or undesirable lap-splice locations could affect the resistance or deformation capacity. Having identified the expected mechanism, the designer must estimate the displacement capacity associated with the mechanism. This requires consideration of both element deformation capacity and the system displacement shape. In addition, care may be required to properly account for higher mode effects on the likely mechanism and local deformation demands. The considerations required for the identification of the plastic mechanism, shear and displacement profiles for various structural systems have been an objective of the 2010-2013 project and developments made will be reviewed in Section 4.

![Image](image.png)

**Figure 1.** Overview of the Direct Displacement-Based assessment Approach (figure taken from Sullivan and Calvi, 2013).
Once the shear and displacement profiles at peak response have been established, the equivalent SDOF system characteristic displacement capacity, \( \Delta_{\text{cap}} \), effective stiffness, \( K_e \), effective mass, \( m_e \), and (if necessary) effective height, \( H_e \), can be found using the following equations:

\[
\Delta_{\text{cap}} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \quad (1)
\]

\[
K_e = \frac{V_b}{\Delta_{\text{cap}}} \quad (2)
\]

\[
m_e = \frac{(\sum m_i \Delta_i)^2}{\sum m_i \Delta_i^2} \quad (3)
\]

\[
H_e = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \quad (4)
\]

where \( \Delta_i, m_i \) and \( h_i \) are, respectively, the displacement, seismic mass and height associated with level \( i \) of the structure, whereas \( V_b \) is the base shear resistance of the equivalent SDOF system at the system displacement value of \( \Delta_{\text{cap}} \) (see Figure 1c).

The effects of energy dissipation and non-linear response are accounted for via the use of an empirical ductility-dependent scaling factor, \( \eta \), which is divided into the equivalent SDOF system characteristic displacement capacity to give \( S_{d,el} \), the equivalent elastic spectral displacement capacity for the same effective period, as shown:

\[
S_{d,el} = \frac{\Delta_{\text{cap}}}{\eta} \quad (5)
\]

As such, the \( \eta \)-factor represents the ratio of the inelastic displacement demand to the elastic spectral displacement demand at the effective period. Traditionally, this ratio is obtained as the combination of a ductility-dependent equivalent viscous damping ratio together with a damping-dependent spectral displacement scaling expression. However, as explained in Pennucci et al. (2011), the scaling factor (also known as a spectral displacement reduction factor) can alternatively be established directly as a function of the ductility demand. In either case, current expressions for the \( \eta \)-factor are empirical, calibrated to fit the results of numerous non-linear dynamic analyses on SDOF systems, and this has been a research activity during the current project, as explained in further detail in Section 4.

With the equivalent elastic spectral displacement capacity established for the assessed effective period, the earthquake intensity expected to cause the limit state to be exceeded is then established as shown in Figure 1d. The engineer, in consultation with the client and local code requirements, can then decide whether the seismic risk is acceptable or whether retrofit is required. Note that the assessed earthquake intensity is expressed in Figure 1d as a probability of exceedence in 50 years. As pointed out by Priestley et al. (2007), this form of the assessment procedure therefore provides information on the probability that a specific limit state is exceeded and for the case shown in Figure 1d it would have been deduced that the probability of exceeding the limit state during a 50 year period is around 29%. Clearly, an
estimate of the probability of exceeding a certain limit state may be more useful than a simple pass-fail assessment approach that will fail to highlight the severity of any problems, if they exist. Nevertheless, as explained in Section 4, the probabilistic considerations made within the DBA procedure of Figure 1 can be improved.

A general benefit offered by the DBA procedure described above is that the engineer is required to consider the likely mechanism that will form and arrive at an estimation of the likely displacement capacity of the structure, considering both local and global deformation capacities. Such considerations are likely to improve the accuracy of seismic assessments. However, guidelines provided in Priestley et al. (2007) were relatively limited focusing on the general considerations required and without providing detailed guidelines for different structural systems. Such observations have motivated research into the DBA of a range of structural typologies, as will be explained in the next section.

3 RESEARCH STRUCTURE

The main objectives of research Line 2 of the RELUIS 2010-13 project have been to develop the general principles and detailed rules of the Displacement-Based seismic Assessment approach. In order to achieve these general objectives, the nine research areas listed in Table 1 have been identified and assigned to different Italian Universities who have strong competencies in the specific research areas. Each research area refers to a different structural typology, such that guidelines for DBA of the main structural typologies that exist in Italy can be developed.

<table>
<thead>
<tr>
<th>Research Area</th>
<th>Responsible University</th>
<th>Research Leader</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. General DBA aspects</td>
<td>Pavia</td>
<td>Calvi &amp; Sullivan</td>
</tr>
<tr>
<td>2. RC Buildings</td>
<td>Bologna and Pavia</td>
<td>Benedetti and Sullivan</td>
</tr>
<tr>
<td>3. Pre-Cast RC Buildings</td>
<td>Bergamo</td>
<td>Riva</td>
</tr>
<tr>
<td>4. Masonry Buildings</td>
<td>Genova and Pavia</td>
<td>Lagomarsino and Magenes</td>
</tr>
<tr>
<td>5. Steel and Composite Structures</td>
<td>Naples Federico II &amp; Pisa</td>
<td>Della Corte and Salvatore</td>
</tr>
<tr>
<td>6. Timber Structures</td>
<td>Trento</td>
<td>Zanon &amp; Piazza</td>
</tr>
<tr>
<td>7. Bridges</td>
<td>Basilicata &amp; Pol. of Milan</td>
<td>Cardone and Petrini</td>
</tr>
<tr>
<td>8. Retaining Structures</td>
<td>Perugia</td>
<td>Pane</td>
</tr>
<tr>
<td>9. Foundations &amp; SSI</td>
<td>Polytechnic of Milan</td>
<td>Paolucci</td>
</tr>
</tbody>
</table>

The overall programme followed by the research line is shown in Figure 2. It is shown that in the first year the project aimed to select case study structures with subsequent consideration of typical mechanisms and deformation limits. The second year aimed to assess each structure using both traditional and displacement-based assessment procedures, with verification of performance via non-linear dynamic analyses. In the final year the research should include improvement of the displacement-based assessment guidelines and re-evaluation of their performance through the use of advanced non-linear analyses or experimental data. In addition, Direct DBA guidelines should be prepared together with a draft model code.
While Figure 2 does provide a good overview of the research programme for the research line, it should be noted that the state-of-the-art varies greatly from one structural typology to another and as such, while this overall research programme is representative of the activities programmed for each research area, the specific objectives of some research units varied slightly. This is particularly true for foundation systems and as such, the activities of the Polytechnic of Milan focused on developing improved means of estimating the stiffness and equivalent viscous damping of foundations, and then incorporating these developments within the DBA of bridge structures. In addition, the Polytechnic of Milan has assisted in providing a software for the selection of displacement-spectrum compatible accelerograms.

4 MAIN RESULTS

The project has successfully made a number of developments to the DBA procedure which are presented in detail in the report edited by Sullivan and Calvi (2013). The following subsections first provide an overview of the general developments to the assessment approach, which is then followed by a description of the progress made for specific structural typologies.

4.1 Improved probabilistic considerations

The procedure explained in Section 2 (after Priestley et al. (2007)) provides an indication of the probability of exceeding the assessment limit state. However, the estimated probability appears to neglect the impact of uncertainties in the demand and capacity. As such, during the course of the 2010-2013 RELUIS project steps were taken to improve the probabilistic considerations being made in the DBA process.
There will be many uncertainties facing the seismic assessment of a structure. One of the most significant sources of uncertainty will clearly be the ground motion intensity but for existing buildings, characterisation of the structure might be considered equally uncertain. In probabilistic assessments, uncertainties tend to be classified as either aleatoric or epistemic and there are various means of dealing with them, as discussed in fib Bulletin 68 (fib, 2012) and elsewhere. Note that in the fib Bulletin a critical general discussion of probabilistic methods is provided. Reviewing the general DBA procedure proposed by Priestley et al. (2007) and described in the previous section, it is clear that the effects of uncertainty are not incorporated within the probability estimate obtained at the end of the process (as per Figure 1d). In this project it is proposed that a simple means of accounting for uncertainty within the DBA procedure (both for what regards the demand and the capacity) is to use the SAC-FEMA approach of Cornell et al. (2002) simplified in line with the recommendations of Fajfar and Dolsek (2010). The SAC-FEMA approach suggests that the probability, \( P_{LS,x} \), of exceeding a given limit state can be established with an \( x \)-confidence level according to:

\[
P_{LS,x} = \bar{H}\left(S_{a,C}\right)C_H C_f C_x
\]

(6)

where \( \bar{H}(S_{a,C}) \) is the median value of the hazard function at the seismic intensity \( S_{a,C} \), that causes a selected limit state to develop, \( C_x \) varies as a function of the confidence level desired, \( C_f \) accounts for the dispersion in demand and capacity and \( C_H \) transforms between mean and median hazard values. Fajfar and Dolsek (2010) point out that this calculation is considerably simplified if it is assumed that mean and median hazard values are approximately equal (such that \( C_H = 1.0 \)) and that a 50% confidence level is sufficient (such that \( C_x = 1.0 \)). Adopting these simplifications, the probability of the exceedence of a given limit state \( P_{LS,x} \) with a 50% confidence level can be obtained as:

\[
P_{LS,x} = \bar{H}\left(S_{a,C}\right)C_f
\]

(7)

where the symbols have been defined above. Note that in the context of DBA, the median value of the hazard function \( \bar{H}(S_{a,C}) \) expected to cause a selected limit state to develop can be considered equivalent to the probability value that is being identified in the general DBA approach explained in Section 2. As such, improved consideration of uncertainties in the DBA process only requires evaluation of the dispersion factor, \( C_f \).

Cornell et al. (2002) report that the \( C_f \) factor intended to account for dispersion (uncertainty) in demand and capacity can be computed, assuming log-normal distributions of demand and capacity, as:

\[
C_f = \exp\left(\frac{k^2}{2b^2}\left(\beta_{DR}^2 + \beta_{CR}^2\right)\right)
\]

(8)

where: \( b \) is a constant that relates the Engineering Demand Parameter (EDP) to the intensity measure and is typically taken as 1.0 (but in reality it should be updated as part of future research to account for different structural typologies); \( k \) is a constant (with values of around 2.0 or 3.0 typical in Italy) used in a power expression to relate the hazard with a probability of exceedence; and \( \beta_{DR} \) and \( \beta_{CR} \) are dispersion measures for randomness in demand and capacity respectively. Fajfar and Dolsek (2010) report that reliable data on dispersion is not yet
available and they used a value of \((\beta_{DR}^2 + \beta_{CR}^2) = 0.2025\). The more recent ATC-58 (2011) document provides many different values of dispersion to account for different phenomena. Using \(k=2.0, b=1.0\) and the dispersion values of Fajfar and Dolsek (2010), one finds from Eq. (7) that the estimated probability of exceeding the key limit state is 1.5 times that estimated without account for uncertainty. This gives an indication of the effect that accounting for uncertainty can have on the assessed probability and note that, formulated in this way, it always leads to an increase in the likelihood of exceeding a given limit state. The accuracy of the SAC-FEMA approach is limited (see Aslani and Miranda (2005)) but it does permit consideration of uncertainty and therefore its implementation within the DBA procedure is considered to be a useful development that could be implemented into national codes to help engineers make a transition into more probabilistic seismic assessment procedures.

4.2 **Relationships between inelastic and elastic spectral displacement demands**

Another general development made during the research project has been to further develop simplified expressions for the evaluation of residual and maximum displacements of structures subjected to seismic actions. These expressions can be of help to the seismic design of new structures and in particular the seismic verification of existing structures. In the last year the analysis database has been increased to include systems with the following features:

- hysteretic cycles (Takeda, Bilinear, Flag, SINA)
- 10 structural periods (0.1s, 0.2s, 0.3s, 0.4s, 0.5s, 0.6s, 0.8s, 1s, 1.5s, 2s, 2.5s, 3s, 3.5s, 4s)
- 14 levels of lateral resistance of the structure (values between 2.5% and 50% of the vertical load)
- 4812 real accelerograms.

The characteristics of the systems were chosen to represent a wide range of possibly existing structures. The high number of accelerograms was kindly provided by Dr. Stafford (Imperial College London). These records are of high quality and have already been suitably filtered and used with success in previous studies (Stafford et al. 2008). The huge amount of data generated for this study from about 2.7 million non-linear dynamic analysis was synthetically contained in a few hundreds of megabytes of data, where each maximum displacement and residual displacement is related to the system and accelerogram input that generated them. This database has been used to investigate the maximum displacement and the residual inelastic displacement.

An example of the value of the results obtained in this work is represented by the following figure, which shows best-fit lines obtained for the spectral displacement reduction factors (Figure 3a) and median values of the ratio of the residual to maximum displacement (Figure 3b) for different hysteretic models. A full summary of the results obtained are included in Sullivan et al. (2013).
4.3 Extending the assessment procedure for the quantification of monetary losses

The research project has made some progress in extending the methodology to permit simplified estimation of direct monetary losses, simplifying and adapting the PEER framework. This started with the proposal by Sullivan and Calvi (2011) that a simplified displacement-based building-specific loss assessment could be undertaken using simplified loss models. This idea was extended by Welch et al. (2012) with the proposal of a four-point loss model, such as that shown in Figure 4a, and in the last year of research it was successfully applied to another case study building with results published in Welch et al. (2014).

In the approach by Welch et al. (2014) the probability of exceeding four key limit states are assessed using the DBA procedure, with uncertainty in the demand and capacity accounted for using a simplified form of the SAC-FEMA approach (described in Section 4.1). Direct losses due to deformations (storey drifts) and accelerations at the selected limit state are computed using EDP-DV functions (see Figure 4b) proposed by Ramirez and Miranda.
(2009). The total expected loss is then computed by integrating the loss model of Figure 4a. By limiting the seismic assessment to four points (limit states) the amount of work required to obtain a loss estimate is relatively limited and the results obtained for two case study buildings (see Welch et al. 2012), suggest that the procedure is promising. However, for assessment of existing structures in Italy, there remain a number of significant uncertainties, not least of which is the cost to be associated to different damage states. As such, it is considered that while the research project has permitted good conceptual development of a simplified building-specific loss assessment approach, further research is required in order to make the new tools applicable in Italy.

4.4 Selection of spectrum-compatible accelerograms

The Polytechnic of Milan also contributed to general developments via the improvement of the REXEL-DISP software, developed in cooperation with the University of Naples Federico II for the selection of displacement-spectrum compatible ground motions and available in the ReLUIS web site (www.reluis.it). A detailed description of the research done is provided in Paolucci et al. (2013a). This activity lead to the publication of Smerzini et al. (2013) and saw a number of modifications and improvements to the current version of REXEL-DISP (v1.2), where a number of additional features were included, namely:

- Re-processing of records: raw acceleration time histories were re-processed relying on the procedure used to process records in the Italian strong-motion database ITACA, with special care to better define the filter bounds and to ensure compatibility of corrected records, in the sense that single and double integration of the corrected accelerograms produce velocity and displacement time series with zero initial conditions and without unphysical baseline trends.
- Enlarge the number of records: an important set of new records was added, with special care to near-field conditions, including the Emilia earthquake sequence and an updated set from the Christchurch earthquake. The SIMBAD database presently consists of 467 three component records from 130 earthquakes worldwide. Most records come from Japan (47%), Italy (18%), New Zealand (17%), and USA (9%), with minor contributions from Greece, Turkey, Iran and other European countries (9%).

The new tools available for record selection should help practicing engineers interested in undertaking non-linear dynamic analyses for the assessment of seismic performance.

4.5 Displacement-based assessment of RC structures

Research undertaken into RC structures has concentrated on different aspects: study and application of the general DB assessment procedure for bare frames with particular reference to the prediction of the collapse mechanism; nonlinear static and dynamic analyses of different types of infilled multi-storey frames for studying the collapse mechanisms and the displacement profiles; proposal of a new DB assessment procedure for infilled RC frames. The findings of this part of the research are reported in detail by Landi and Benedetti (2013). The work included examination of a number of bare RC frame case study structures: a five storey three bay frame, a five storey five bay frame and a ten storey three bay frame. Both DBA and pushover analyses were conducted considering the formation of a global collapse mechanism. For all the analysed frames comparisons between the DBA procedure and nonlinear dynamic analysis results were made. Moreover an approach has been proposed...
based on limit analysis for the rapid prediction of the collapse mechanism. The application of this approach has provided results in agreement with those of pushover analyses. Furthermore the application of the DBA procedure to infilled RC frames has been studied. According to the Displacement Based procedure, the seismic assessment of existing buildings requires the definition of the damping of the examined structure. To this purpose in the second year an extensive campaign of nonlinear dynamic analyses had been carried out. From the results of these analyses it was possible to derive proper ductility-damping laws for infilled frames which could be used in the seismic assessment, without knowing in detail the real response of the infilled frames in terms of stiffness and strength. Moreover in the second year an equivalent strut model was calibrated on the basis of comparisons with available experimental results relative to monotonic and cyclic loading cases. This model was applied for analysing a five storey RC frame with and without the presence of masonry infills. Pushover analyses have been performed in order to obtain the response in terms of base shear-top displacement and to evaluate the configuration at collapse and the displacement profile. In the third year, for the same structures, nonlinear incremental dynamic analyses were also performed. Through the execution of incremental dynamic analyses it has been possible to evaluate the average peak ground acceleration at collapse: for the bare frame 0.544g and for the infilled frame 0.767 g. As seen in Figure 5, the predictions obtained via the IDA approach correlate very well.

Overall, the research undertaken through the course of this three year research project has lead to the formation of useful guidelines for the assessment of RC structures, and it has been shown that for existing RC structures, the DBA approach can be an effective and practical assessment tool.

4.6 Displacement-based assessment of pre-cast concrete structures

Guidelines for the displacement-based assessment of pre-cast concrete structures were developed by Torquati et al. (2013) and included examination of both single and multi-story precast (typically industrial) buildings with consideration of the peculiarities of these buildings, related to the influence of the connections between the structural elements and to the high deformability offered by the static scheme mainly identified by columns hinged to the beams.

As reported in Torquati et al. (2013) guidelines have been provided for the definition of the force-displacement and moment-rotation relationship of the joints between pre-cast beam-
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column elements, utilizing formulations available in the literature. Furthermore, the formulation for the yielding curvature of reinforced concrete columns provided by Priestley (2003) has been recalibrated with a series of moment-curvature parametric analyses on different cross-section types, in order to provide more adequate expressions for the columns used in this type of construction. The high deformability which characterizes this type of structures can significantly intensify the second order effects, and for this reason, a study on P-Δ effects has also been performed as reported in Torquati et al. (2013).

Considering the general displacement-based assessment procedure for precast buildings, Torquati et al. (2013) developed two methods for the evaluation of the inelastic displacements profile of the structure: a Pushover Method (PM) and an Equivalent Column Simplified Method (ECSM). The effectiveness and applicability of the proposed DBA procedure for existing precast buildings has been evaluated considering two case studies and has been compared with other seismic assessment methods available. The case studies considered are representative of a three-story precast building, whose seismic vulnerability is evaluated with different analysis methods and then compared with each other in terms of the estimated PGA required to reach the assessment limit state. The assessment methods utilised in this work include the: Pushover (N2 Method), DBA – Pushover Method (DBA-PM), DBA – Equivalent Column Simplified Method (DBA-ECSM), and an Incremental Dynamic Analysis (IDA) approach. The beam-column connections are considered as dowel connections with neoprene cushions at the supports. The following figure shows one of the case study structures examined (case study A).

In order to evaluate the influence of the stiffness of the structure during the calculation of the PGA associated with the ultimate limit state, a second case study was subsequently evaluated, assuming column cross-sections of 60x60cm and a stiffness of the beam-column connection four times greater at the same strength (case study B). IDA were carried out to evaluate the effectiveness of the proposed procedure, and the limit state PGA has been obtained as the mean value of the results of 7 selected ground motions spectrum compatible in displacement with the target spectrum (Serra Pedace – Cosenza): $a_g=0.276g$, $F_0=2.438$, $T_c^*=0.374$, $S=1.296$, $C_v=1.453$. 

![Figure 6. Elevation (a) and connection detail (b) of case study A pre-cast building (after Torquati et al. 2013).](image)
The analyses on the case studies provided the results shown in Table 2:

<table>
<thead>
<tr>
<th>Case Study A – PGA (g)</th>
<th>Case Study B – PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No P-delta</td>
<td>With P-delta</td>
</tr>
<tr>
<td>Pushover – N2</td>
<td>0.274</td>
</tr>
<tr>
<td>DBA - PM</td>
<td>0.319</td>
</tr>
<tr>
<td>DBA – ECSM</td>
<td>0.276</td>
</tr>
<tr>
<td>IDA</td>
<td>0.315±0.050</td>
</tr>
<tr>
<td></td>
<td>0.307±0.048</td>
</tr>
</tbody>
</table>

* PGA corresponding to the maximum return period of the site considered: higher PGA values not included in the available site data.

The ultimate limit states associated to the PGA of the previous table refer to the failure of a beam-column connection for case study A, and to the failure of one of the base columns associated with the ultimate rotation capacity for case study B.

Taking as a reference the results provided by IDA, the Pushover-N2 method shows unreliable results: the structural response in case A is underestimated (conservative solution), but it is overestimated in case B. This method does not take into account the effective damping of the system.

Regarding the assessment procedures, conservative results are obtained using the simplified approach DBA-ECSM, which underestimates the PGA of the dynamic analysis by about 15% for case A and about 50% for case B; the approach DBA-PM leads to a good PGA estimation, with about 3% error in both cases.

### 4.7 Displacement-based assessment of masonry structures

Developments for the displacement-based assessment of masonry structures have been made by Cattari and Lagomarsino (2013). After a general review (first year) of the issues to address for the DBA of masonry structures, with respect to both the global and out-of-plane seismic response, the research mainly focused on the global response. Moreover, the issue of seismic assessment has been faced also according to a probabilistic approach. In the second year, both specific modelling tools (multi-linear constitutive laws for masonry panels on phenomenological basis which have been implemented in the Tremuri software) and criteria for the definition of Limit States (a multi-scale approach based on combined checks at element, macroelements and global scales) have been developed. The main aim was to provide tools that could simulate more reliably some of the relevant features of existing masonry buildings, such as weak spandrels, flexible floors, irregularities: factors that greatly affect their seismic response more than in the case of the new buildings. Finally, in the third year, after the validation of such proposed tools, nonlinear parametric analyses on some prototype configurations were performed in order to establish correlation laws between Limit States and some of the entities necessary for the application of DBA (e.g. damping and deformed shapes).

The reliability of the multi-linear constitutive laws (phenomenological basis) for masonry panels implemented in the Tremuri software (that works according to the equivalent frame approach), and that of a multi-scale approach for the definition of Limit States on the capacity curve have been validated through the simulation of the seismic response of a real building located in San Felice sul Panaro and seriously damaged by the May 29, 2012 event. Both non-
linear static and dynamic analyses have been performed. Results highlighted a good agreement with the real response that occurred (see Figure 7); moreover, the damage level simulated (based on the comparison between the maximum displacement obtained from the nonlinear dynamic analysis and the SL thresholds assessed through the multi-scale approach) complies with that observed.

Figure 7. Numerical simulation (with Tremuri program) of a masonry case-study building located in San Felice sul Panaro (from Cattari and Lagomarsino 2013).

The multi-linear constitutive laws developed were found to be capable of simulating different types of hysteretic behaviour as a function of various prevailing damage modes (if flexural, shear or mixed) and two types of masonry panels (pier and spandrel). They constitute a very useful and versatile tool to describe some of the specific features of various recurring global seismic responses of existing masonry buildings (as testified by the damage survey) and to characterize them in terms of different hysteretic properties, ductility and collapse shapes, as seen in Figure 8.

Figure 8. Capability of multi-linear constitutive laws to simulate (through nonlinear cyclic static analyses) some of the specific features of existing masonry buildings (Cattari and Lagomarsino 2013).
Once validated, the aforementioned tools have been adopted for the subsequent research activities that undertook nonlinear parametric analyses on some prototype configurations. The final aim of this activity was to define proper correlation laws - applicable to the case of existing masonry buildings - between Limit States and some of the entities useful for the application of the DBA (e.g. damping, story drift, deformed shapes). The analyses have been carried out as a function of: five prototype configurations; two classes of masonry type; flexible or rigid floors; with or without the presence of some recurring strengthening interventions (e.g. tie-rods). The results achieved allow to differentiate such laws as a function of factors related to the irregularity and floor stiffness. For example, results highlighted that, in the case of flexible floors, the definition of a global limit state tends to be affected more by the checks performed at element and macroelement (e.g. masonry walls) scales, that tend to move the limit state position further back on the capacity curve than in the case of rigid floors, as indicated by Figure 9.

Figure 9. Example of the influence of the floor stiffness on the definition of limit state within the pushover curve: (a) flexible and (b) rigid floors (after Cattari and Lagomarsino 2013).

4.8 Displacement-based assessment of steel structures

Displacement-based seismic assessment guidelines for steel structures have been developed by Della Corte et al. (2013a, 2013b). The percentage of steel structures within the whole Italian building stock is fairly low. However, a significant portion of the industrial buildings in Italy are made of steel, and the risk of business interruption due to earthquakes may be significant in case of industrial buildings. This last statement is strengthened by consideration of the larger deleterious effects of damage to one or few buildings on the complete production chain in a given industrial sector.

As a result of the above, the research conducted by Della Corte et al. (2013a, 2013b) was focused mainly on issues related to modelling and response analysis of steel structures typical of industrial buildings. However, a more general study was carried out with reference to bolted end-plate connections which are frequently encountered in any type of steel structure. The research into beam-column joints by Della Corte et al. (2013a) lead to the realisation of simple methods to characterize the mechanical behaviour of bolted end-plate beam-to-column joints. The method currently adopted by Eurocode 3 is well known and named the “component method”. It decomposes the joint response into the assemblage of response of
simpler components. The stiffness and strength of each component is estimated analytically and all the components are then assembled together based on simple kinematic assumptions. This is an analytical and conceptually general method but requires time for implementation and does not provide general and quick information about trends of mechanical characteristics with key joint parameters. Through parametric analysis of typical beam-to-column joints and using the component method, the possibility to derive simple closed-form equations to evaluate the stiffness and strength of preselected joint configurations has emerged. Such closed-form equations could provide designers and analysts with simple and ready to use tools to evaluate the key joint mechanical characteristics. Differences between experimental results and theoretical predictions have been shown to be appreciably larger than differences between the component method and the simplified equation predictions. It is concluded that further research is justified, because the component method is laborious to use and requires many calculations whereas the alternative method being formulated allows one to make simpler (faster) calculations that should provide reasonable accuracy. Clearly there is a trade-off between simplicity, accuracy and generality, and future research should aim to strike the right balance between these factors.

The other important area of research for steel structures included the detailed examination of an industrial case-study building, with results published by Della Corte et al. (2013b). It was found that a careful evaluation of the column base connections was paramount for the behaviour of the whole system. Stiffness, strength and deformation capacity of such connections need assessment in order to evaluate the structural performance. Such an assessment of connection characteristics can be difficult because of the differences in geometry between existing connections and standard types covered by current Structural Codes. In addition, even in the case of connections conforming to standard types, no explicit analytical method is currently available to evaluate the deformation capacity. As far as strength and stiffness is concerned, extensions/adaptations of available methods have been proposed, based on simple mechanical principles. The validity of such methods needs evaluation through experimental tests. Another important observation made by Della Corte et al. (2013b) was that some mechanisms could only be reproduced in non-linear dynamics analyses when 3D models were used, suggesting that some guidelines for modelling requirements could be developed as part of future research.

4.9 Displacement-based assessment of composite steel-concrete structures

Developments for the displacement-based seismic assessment of composite steel-concrete structures have been made by Morelli and Salvatore (2013). The University of Pisa unit group developed a beam-to-column joint cyclic model starting from the component method proposed by Eurocode 3 (for steel structures) and Eurocode 4 (for composite structures). Each joint component has been modeled by a suitable force-displacement or moment-rotation relationship, while the concrete slab was schematized by fiber elements in order to model, as accurately as possible, the non-linear behavior and the crushing of the concrete (Figure 10). The characteristics of each joint component was calibrated against the results of experimental tests executed on substructures in order to evaluate the capacity of the model to represent the actual behavior of the joint.

Starting from what was presented in Braconi et al. (2007) a component model was developed to reproduce the observed response of the entire sub-assemble beam-to-column test specimens, including beam and column flexural behaviour. In the model, the connections between the beam endplates and the column flanges were represented by equivalent T-stubs localized at top and bottom beam flanges. As shown in Figure 10 for the interior joint, the
model accounted for the response of: the unconfined concrete in compression (1), the confined concrete in compression (2), the lower T-stub (3), the upper T-stub (4), the wire mesh (6), the reinforcing bar (7) and the concrete in tension (11). Two rigid elements were introduced to simulate the connection between the composite beam and the joint. A fiber representation was adopted for the concrete slab in compression and in tension to adequately capture the non-uniform stress distribution over the slab thickness.

The model shown in Figure 10 was accurately calibrated to the results obtained from cyclic tests on joint sub-assemblages executed in Pisa. The development of new guidelines for the modelling and non-linear analysis of beam-column joints composite structures is considered to represent a valuable development for displacement-based assessment since it will permit improved understanding and evaluation of the force-displacement response of composite structures.

4.10 Displacement-based assessment of timber structures

Guidelines for the displacement-based seismic assessment of timber structures have been provided by Loss et al. (2013). The approach proposed by Loss et al. (2013) is based on the definition of simplified models for calculating the structural capacity, specific for the most likely failure mechanism and the reference limit state (ultimate and serviceability). These models allow evaluation of the displacement, force and energy dissipation capacity of shear wall elements, where the concept of “shear walls” extends to both framed and cross-laminated timber panels. Models have been developed both for the serviceability and ultimate limit states. The capacity of the individual wall elements is extended to a known structural system through simple analytical models which allow to estimate the limit displacement and deflection of the structure for a given structural mechanism. The deflected global shape can be
estimated using simplified formulas similar to those proposed for displacement-based design of timber buildings. The most critical structural failure mechanism is identified with the aid of so-called mechanism indices, which suggest the most likely failure mechanism based on the mechanical-geometrical properties of walls and connections and on the load configuration. Figure 11 illustrates two different mechanisms that can be expected for timber framed wall structures.

The proposed method has been validated, first, via numerical simulation and, then, through a comparison with the outcomes of laboratory results. In addition, the procedure has been validated on a real case study timber building. Figure 12 compares displacement profiles obtained from non-linear dynamics analyses with those predicted using DBA. The method in the present form is directly applicable to buildings where second order effects and torsional effects can be neglected. In addition, the effectiveness of the procedure is conditional to the possibility of identifying the most critical failure mechanism or at least the most likely.

In a strict sense, the assessment method proposed by Loss et al. (2013) has been conceived for timber buildings which are regular in plan and elevation. Nevertheless, the method could be applied to buildings which do not satisfy regularity criteria, provided that the engineer carefully evaluates the analysis results.
4.11 Displacement-based assessment of bridges

A comprehensive procedure for the Direct DBA of existing bridges has been developed by Cardone and Perrone (2013). The fundamental step of the proposed procedure is the definition of the so-called Performance Displacement Profile (PDP) of the bridge, corresponding to the inelastic bridge deformed shapes associated with the attainment of selected Damage States (DS’s) in some critical elements of the bridge.

In the work by Cardone and Perrone (2013), the Displacement Limits associated with different DS’s of the piers, abutments, joints, bearing devices and shear keys have been defined and comprehensively discussed. Moreover, a number of alternative approaches for the definition of the PDP have been examined, including: (i) Displacement Adaptive Pushover (DAP) analyses, (ii) Effective Modal Analysis (EMA), (iii) analysis of Individual Pier-deck Models (IPM), for bridges with simply supported independent adjacent decks and (iv) rational analysis, for continuous deck bridges. Finally, several aspects related to bridge modelling, including the selection of a suitable skeleton curve and effective damping ratio for each structural member, have been discussed.

Cardone and Perrone (2013) applied the proposed DDBA procedure to a set of eleven bridge configurations, differing in pier layout, deck type and bearing device characteristics. The predictions of the proposed DDBA procedure have been compared with the results of accurate NonLinear Response time-History Analysis (NLRHA), carried out on refined numerical models of the bridge, using two sets of seven accelerograms compatible (on average) with given reference response spectra scaled to the PGA values provided by the DDBA procedure for different Performance Levels (PL’s) of the structure. Figure 13 illustrates the model developed to assess the non-linear dynamic response of the Kavala bridge, from the Greece Egnatia motorway.

![Figure 13. Analytical model of the Kavala bridge (from Cardone and Perrone 2013).](image)

The comparisons between DDBA predictions and NLRHA results reported by Cardone and Perrone (2013) confirm the good accuracy of the proposed procedure in predicting the PGA values associated with slight-to-severe DS’s of piers, bearing devices and abutments. In all the examples considered, indeed, the DDBA procedure was found to correctly identify the
critical element of the bridge, where a first given DS is reached. Moreover, the PDP of the bridge assumed in the analysis was seen to be in good accordance with the maximum deformed shape of the bridge obtained from NLRHA. Figure 14 provides a sample of results obtained, comparing the peak displacements assessed by different methods for the Kavala bridge at the moderate (repairable) damage limit state.

4.12 Allowing for soil-structure interaction

An iterative pseudo-static seismic assessment procedure for multi-span reinforced concrete bridges, based on the DDBA procedure coupled with Non-Linear Soil-Structure Interaction (DDBA + NLSSI) was developed by Paolucci et al. (2013b) during the project, which allows the introduction of non-linear dynamic soil-structure interaction effects using an equivalent linear visco-elastic approach. The procedure is iterative in order to arrive at a displacement profile (that includes effects of foundation deformations) and an internal force distribution that is consistent with the inertia force distribution that the displacement shape implies. The procedure has been developed for SDOF (single pier) and MDOF (whole bridge) systems subjected to longitudinal or transversal seismic action and has been applied to hypothetical SDOF/MDOF systems and to a real existing bridge; the Fiumarella Bridge.

For the validation of the DDBA+SSI procedure, IDA were used with SSI being taken into account using a macro-element model. A preliminary study aimed at validating the macro-element with experimental results and with numerical results obtained with another software named CHOPIN_F10 was performed.

During IDA, for each level of seismic intensity and corresponding scale factor considered, the maximum top displacement at each pier was recorded and compared with its displacement capacity obtained from the DDBA+SSI procedure. When one of the pier tops reached the displacement capacity, the corresponding scale factor was recorded as a Capacity/Demand ratio and the envelope of the pier/abutment displacements was collected as the “bridge displacement profile”. The mean value of Capacity/Demand (C/D) ratios for the considered accelerograms was taken as the global C/D ratio for the structure being assessed.

The comparison between the C/D ratios obtained from DDBA+SSI procedure and from the IDA has given a quantitative indication of the procedure's accuracy. Moreover its accuracy has also been evaluated comparing the bridge displacement profile expected based on DDBA+SSI with the average of the displacement envelopes collected for the considered accelerograms.

For the cases studied, the errors in terms of C/D ratio were found to be in general lower than 5% (only in 1 case it reached 7.2%). The errors in terms of displacement profile are always
lower than 20% for the piers but they can reach very high values (up to 300%) for the abutments indicating a limit in the procedure in describing the abutment behaviour.

To gain better insight into the likely effects of SSI for bridge pier response, the assessment of four bridge piers designed with a minimum consideration of horizontal loading (design against wind loading) were also considered for assessment of the seismic demand against earthquakes with different probability of occurrence. Both a simplified procedure and non-linear time history analyses were considered to evaluate the role of the non-linear behaviour of the foundation system on the overall seismic assessment of the structure. For this purpose three cases have been investigated (all of them with non-linear behaviour of the superstructure): 1) fixed-base; 2) flexible foundation with linear behaviour; 3) flexible foundation with non-linear behaviour. The parametric analyses highlighted that for low-rise piers, tentatively with height $H < 15$ m, the role of non-linear foundation response may indeed be critical in reducing the ductility demand and in changing assessment outcomes, with respect to piers with fixed (or linear-elastic) foundations.

In conclusion, the research into foundations and SSI has been focused on the case of bridge structures and has shown that SSI effects can be very significant. The work has also indicated that a DBA approach can provide good indications of the effects of SSI and should therefore be developed further as part of future research, developing tools to deal with other foundation typologies (e.g. piled foundations) and other structural systems (e.g. buildings).

### 4.13 Retaining structures

Proposals for the displacement-based seismic assessment of retaining structures have been developed in this project by Cecconi and Pane (2013). The research undertaken included the development of a normalization-procedure for simplified application of the displacement-based method to retaining structures. The proposed procedure provides non-dimensional charts and equations for the seismic thrust, thus allowing the designer to skip the mass discretization and the associated displacement profile, and to use simple (non-dimensional) reduction factors of the seismic thrust as a function of the design displacement. The procedure is aimed at analysing the comparison between the displacement capacity vs. the displacement demand and, at the same time, at evaluating the proper design solution.

Different strain fields were considered in order to improve and rationalize the definition of the equivalent damping ratio for the whole soil/retaining structure system. Cecconi and Pane (2013) developed a simple equation for the equivalent damping of a retaining wall system as a function of the peak displacement by assuming that the non-linear stress-strain behaviour of the soil under monotonic loading could be described by the Ramberg-Osgood model with the Masing criterion used to model the hysteretic behaviour upon cyclic loading. The resulting damping relationship is recommended by Cecconi and Pane (2013) for use with all cantilever retaining structures in non-plastic soils.

The research also investigated the general applicability of DDBA to cantilever retaining structures. In the initial stage, and for different limit states, the assessment displacement (occurring at the top of the wall) was defined based on available structural details. A plastic hinge is assumed to form at the RC wall section where the curvature attains its maximum value. The CUMBIA code (see Priestley et al. 2007) was used to build a simplified bi-linear moment–curvature relationship and then the force-displacement response was calculated. The displacement profile for the whole system was then tested by means of soil/structure interaction numerical analyses, by varying the relative stiffness of the components. The comparison between the capacity displacement and the demand displacement or, in other
words, the evaluation of the C/D ratio represents the vulnerability of the system for the considered limit state.

The assessment procedure was applied by Cecconi and Pane (2013) to different case-study retaining structures of the form indicated in Figure 15a, embedded in cohesion less medium-dense sands. The results of the assessment procedure for one of the case study walls are summarised in Figure 15b, which shows the assessed displacement demands and the exceedence probability in 50 years for different limit states.

![Figure 15. Embedded retaining wall system (a) and results of displacement-based seismic assessment of a case-study retaining wall system (b) (from Cecconi and Pane 2013).](image)

5 DISCUSSION

Reviewing the results described in Section 4, it is evident that the project has significantly improved the state of the art for displacement-based seismic assessment. A number of useful tools and guidelines have been developed that are expected to provide the practicing community in Italy valuable insight into the critical factors influencing the seismic vulnerability of existing structures. Developments for certain structural typologies (e.g. bridges) have been more significant than for others (e.g. retaining walls) but it is considered that this essentially reflects the different amounts of research done previously for certain structural typologies and not for others. Overall the project has been very successful in developing guidelines for the displacement-based seismic assessment of structures, that have been published in a detailed research report, available on-line (Sullivan and Calvi 2013). The project also saw the formation of a first draft of a model code for displacement-based assessment. However, it was not possible to develop this model code to a point that could be published, owing to the large number of areas that were identified as requiring further research. Nevertheless, the large number of valuable developments to the DBA method made during this project implies that the publication of a model code for displacement-based seismic assessment should be possible in the near future.

6 VISIONS AND DEVELOPMENTS

Tools for accurate but simple seismic assessment of structures are particularly important for Italy owing to the large stock of existing buildings and the reasonably high levels of
seismicity that exist in different parts of the country. Such tools and guidelines would be all the more valuable if their application could instill in engineers a better understanding of the fundamental concepts of seismic assessment and the features of a structural typology that are likely to critically affect the seismic risk. These observations underline the real value in considering the adoption and further development of displacement-based design and assessment methods in Italy.

Considering various technical points raised in the literature, it becomes apparent that there are a number of inconsistencies between seismic design and assessment methods in current codes, so much so that it may even prove difficult to demonstrate that a newly realized structure designed using force-based methods actually meets current code assessment criteria. The application of displacement-based design and assessment procedures overcomes such issues and additionally provides engineers with a better sense of the role that structural proportions, material properties, member detailing, and capacity design concepts all play in the apparent seismic risk. This project has brought the possibility of displacement-based codes one step closer, successfully developing a detailed set of guidelines and tools for the displacement-based seismic assessment of buildings and bridges. Furthermore, this project has evidenced what appears to be an opportunity to extend the displacement-based assessment procedure for simplified assessment of the probability of exceeding key limit states and provide information on more tangible performance parameters, such as monetary losses. This also suggests that a vision for the future should include consideration of how displacement-based methods that provide building-specific loss assessment could guide the identification of effective retrofit strategies, that consider not only up-front construction costs but the whole life-cycle performance. Such observations underline the value gained from this ReLUIS project and suggest that displacement-based methods could play an important role in reducing seismic risk in the years to come.

7 MAIN REFERENCES


Developing Guidelines for Displacement-Based Seismic Assessment


8 RELUIS REFERENCES


Developing Guidelines for Displacement-Based Seismic Assessment


Petruzzelli F., Della Corte G. and Iervolino I. (2011b) “Modelli e analisi preliminari per la valutazione del rischio sismico di edifici industriali esistenti” *Attidel Congresso CTA (Collegio dei Tecnici dell’Acciaio)*, Ischia.


