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DISPLACEMENT-BASED SEISMIC ASSESSMENT AND REHABILITATION OF EXISTING NON-DUCTILE REINFORCED CONCRETE STRUCTURES

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ABSTRACT
Traditionally, seismic assessment and rehabilitation of existing structures has been carried out employing the force-based approach. In recent years, the displacement-based approach has emerged as a favourable alternative since displacement-based parameters better quantify structural and non-structural damage than force based parameters. This paper demonstrates the advantages of the displacement-based approach over the traditional force-based approach for seismic assessment and rehabilitation of existing "non-ductile" reinforced concrete structures. An existing industrial reinforced concrete frame structure is assessed by both approaches, leading to two conceptionally different rehabilitation schemes. The advantage of the displacement-based approach for seismic assessment and rehabilitation of existing non-ductile reinforced concrete structures, in terms of better understanding and quantifying of the performance under the design earthquakes, effectiveness and cost efficiency of the resulting rehabilitation scheme, is clearly demonstrated.

Keywords: Seismic assessment; Displacement-based method; Non-ductile structures

INTRODUCTION
There are a large number of existing civil, industrial and transport structures that are located in seismic regions which were not designed and constructed in accordance with modern seismic design codes. In particular, the detailing in these structures normally does not satisfy the ductile detailing provisions specified in modern codes. They are therefore often called "non-ductile" structures although they may possess certain limited ductility. Continuing occupancy of some of these buildings, such as schools, hospitals and essential industrial and transport facilities, requires seismic retrofit to bring them in line with modern standards. The process of seismic rehabilitation of existing structures normally consists of a number of stages including data and information collection, seismic assessment, seismic retrofit design and seismic retrofit implementation. This paper focuses on the choice of the methodology for the seismic assessment and retrofit scheme design.

Traditionally, seismic assessment and rehabilitation have been carried out employing the force-based methodology. This approach has been adopted in Part 1-4 of Eurocode 8 [1] on seismic evaluation, strengthening and repair of existing buildings. In recent years, the displacement-based methodology has emerged as a favourable alternative since displacement-based parameters better quantify structural and non-structural damage than force based parameters. This paper demonstrates the advantage of the displacement-based approach over the traditional force-based approach for seismic assessment of existing "non-ductile" reinforced concrete structures. It employs the non-linear dynamic procedure of FEMA 273 [2] and ATC 40 [3].

An existing reinforced concrete frame structure is assessed by both approaches. Two viable retrofit solution schemes have been proposed on the basis of conclusions drawn from both assessment methods. Cost implications corresponding to both retrofit strategies are then compared. The advantage of the displacement-based approach over the force-based approach for seismic assessment and rehabilitation of existing non-ductile structures, in terms of better understanding and quantifying of the performance under the design earthquakes, effectiveness and cost efficiency of the resulting rehabilitation scheme, is clearly demonstrated.

BRIEF DESCRIPTION OF THE EXISTING STRUCTURE

The existing structure is an industrial reinforced concrete space frame supporting a vessel. It consists of six columns located at the six corners of a hexagon and ring beams at 7.6 m and 17.4 m above the ground level. The diameter of the circle on which the geometric centres of the six columns locate is 7.8 m. Figure 1 illustrates the existing structure and gives dimensions of the beams and columns (in milimetres).

LOCAL SEISMIC HAZARD, THE DESIGN EARTHQUAKE AND THE SEISMIC PERFORMANCE OBJECTIVE

The structure is located in a low to moderate seismic hazard region. A site seismic risk study has indicated that the major seismic hazard arises from a distant large magnitude earthquake. A site peak horizontal ground acceleration of 0.25g has been determined to correspond to a distant Maximum Considered Earthquake (MCE). The probability of exceedance corresponding to the MCE is approximately 2% in 50 years. The MCE ground motion at the site is represented by a 5% damped, elastic response spectrum as shown in Figure 2. The qualitative seismic performance objective is to prevent collapse of the structure but to permit damage to such a degree that it is on the verge of collapse under the MCE.

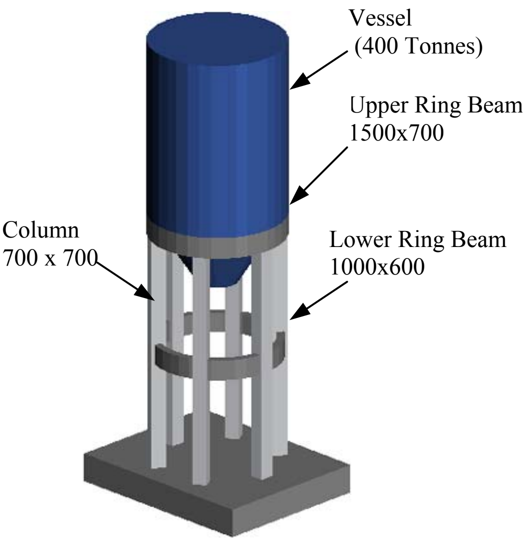

Figure 1: The existing structure

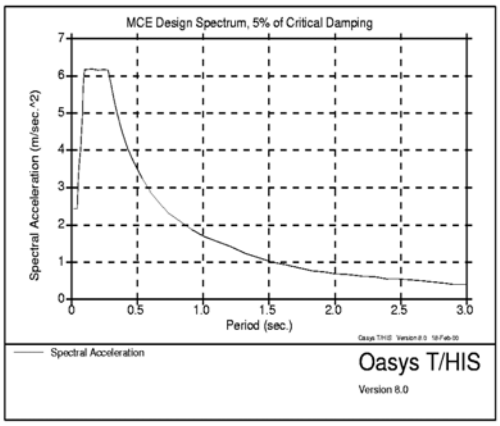

Figure 2: Design earthquake response spectrum

SEISMIC ASSESSMENT AND RETROFIT BY THE CONVENTIONAL FORCE-BASED METHODOLOGY

Overview of the Force-based Methodology

The conventional force-based seismic design and assessment methodology is usually based on a set of code-defined design and detailing rules. It does not require the design engineer to understand the likely behaviour and performance of the structure under the design earthquake. For instance, the component (member) strength capacities are required by seismic codes to be not less than the elastic seismic force demand divided by the force reduction factor, R (termed the structural behaviour factor, q, in Eurocode 8). In addition, the detailing of components is required to comply with the seismic detailing provisions for adequate ductility capacity. Part 1-4 of Eurocode 8 specifies that the design and detailing of an existing structure should be assessed in accordance with Part 1-3 in order to determine the ductility class (DC "H", DC "M" or DC "L") appropriate to the existing structure. Then, an appropriate behaviour factor can be established for the entire existing structure in accordance with Part 1-3 of Eurocode 8.

An existing structure or the design of a new structure is considered satisfactory by virtue of satisfying all relevant seismic code provisions. However, the likely behaviour of the structure designed or assessed by the conventional force-based methodology is not required to be known according to the force-based seismic design and assessment philosophy. The expected performance is only described in broad terms (e.g. life safety, collapse prevention).

Material Strength Values of the Existing Structure

The material strength values of the existing concrete structure have been determined based on design drawings and construction records as follows:

Best estimate concrete cylinder compression strength = 22 MPa
Minimum main reinforcement yielding strength = 400 MPa
Minimum stirrup yielding strength = 240 MPa
Best estimate main reinforcement yielding strength = 1.25 x 400 MPa = 500 MPa
Best estimate stirrup yielding strength = 1.25 x 240 MPa = 300 MPa

Assessment of Component Ductile Detailing

Careful examination of available drawings reveals that the detailing of the beams, columns and the beam-column joints does not satisfy requirements as set out in ACI 318-95 [2] for ductility. Insufficient transverse reinforcements have been provided in these components with the distance between adjacent stirrups exceeding significantly the maximum permitted distance specified in ACI 318 – 95 [2]. Therefore, the existing structure does not satisfy code ductile detailing rules, particularly at the likely plastic hinge zones at the ends of beams and columns and at the beam-column joints.

Assessment of Component Strength Capacities and Elastic Seismic Force Demands

The assessment of strength capacities of the various components of the existing concrete structure is carried out in accordance with ACI 318-95 [4]. The best estimate flexural (deformation-controlled) strength values and the lower bound shear (force-controlled) strength values of various components have been evaluated. These were then modified by the "knowledge factor" $\kappa = 0.75$ according to FEMA 273 [2] and are summarised in Table 1.

The purpose of the knowledge factor is to account for uncertainty in the strength, condition and ductility of elements of an existing structure. The greater the degree of in-situ inspection and material sampling and testing performed the higher the value of κ, to a limit of 1.0 if extremely extensive investigations have been performed.

The component elastic seismic force demands are determined by carrying out a dynamic response spectrum analysis. Both the cracked and the uncracked stiffness of components have been considered in determining the dynamic properties of the structure. In this paper, only elastic seismic force demands corresponding to the cracked component stiffness are presented. Those corresponding to the un-cracked component stiffness are significantly higher. A uni-axial earthquake ground motion input is applied. Comparisons of some key component elastic seismic force demands with the corresponding component strength capacities are summarised in Table 1.

Results in Table 1 suggest that, in the case of using the cracked stiffness, the lower ring beams and the beam-column joints are overstressed in shear by factors of 2.7 and 1.9, respectively. Further, the lower ring beams are overstressed in flexure by a factor of 1.4. The demand-capacity ratio for flexure in the lower ring beams is even higher, being 1.7. Under the excitation of the design earthquake, failure of the existing structure is initiated by the shear failure of the lower ring beams. Since shear failure in beams and joints is brittle, the limited flexural ductility capacities in the lower ring beams and the columns cannot be utilised. Even if the lower ring beams and the beam-column joints had sufficient shear strength, the lower ring beams would still have been overstressed in flexural by a factor of 1.4 or 1.7, depending on whether cracked or uncracked stiffness is used. Considering that the lower ring beams do not satisfy code ductile detailing requirements for ductility in flexure, the existing structure would still be considered as not satisfying modern seismic code requirements.

TABLE 1 SUMMARY OF COMPONENT STRENGTH CAPACITIES AND DEMANDS

Component	Action	Strength before Applying κ	Strength After Applying κ	Elastic Seismic Force Demand	Demand-Capacity Ratio
Column	Axial-Flexure	1875 KNm (best estimate)	1400 KNm	866 KNm	0.62
Column	Shear	593 KN (lower bound)	445 KN	195 KN	0.44
Lower Ring Beam	Flexure	1188 KNm (best estimate)	890 KN	1200 KNm	1.40
Lower Ring Beam	Shear	391 KN (lower bound)	294 KN	800 KN[1]	2.72
Upper Ring Beam	Flexure	2625 KNm (best estimate)	1970 KNm	673 KNm	0.34
Upper Ring Beam	Shear	2100 KN (lower bound)	1574 KNm	425 KN	0.27
Beam-Column Joint	Shear	1530 KN (lower bound)	1150 KN	2200 KN[1]	1.91

Note: [1] determined considering the best estimate flexural strength of beams, taking $\phi = 1.0$

Table 1 also indicates that the MCE elastic force demands of columns and the upper ring beams are lower than their corresponding strength capacities. Therefore, retrofit measures may not be required for the columns and the upper ring beams.

Force-based Retrofit Concept and Costs

The force-based methodology naturally leads to a retrofit concept that upgrades the existing structure in two fronts. Firstly, strengthening the lower ring beams and joints in shear is required in order to prevent brittle failure of these components. Secondly, increasing the transverse reinforcement in the plastic hinge zones of the lower ring beams is required in order to satisfy code ductile detailing requirements. Alternatively, upgrading the flexural strength of the lower ring beams can be carried out such that they are capable of resisting the MCE elastically without the need for ductile detailing. The objective of the force-based retrofit concept is to satisfy code provisions on strength and ductile detailing.

Once this objective is achieved, an appropriate seismic force reduction factor (or structural behaviour factor) can then be established for the existing structure according to the seismic design code adopted. In practice, it is difficult to increase the transverse reinforcement in beams and columns of an existing reinforced concrete structure. On the other hand, it is relatively easy to increase the flexural strength of the lower ring beams by bonding steel plates to their top and bottom surfaces. These beams and the joints can also be strengthened in shear by bonding steel plates to their side faces. This retrofit scheme not only increases the shear strength but also increases the bending strength and stiffness of the lower ring beams. As a result, the columns become relatively weaker than the lower ring beams resulting in an undesirable “weak column-strong beam” mechanism. Furthermore, due to the increased stiffness of the lower ring beams, the fundamental period of the retrofitted structure is decreased, leading to even higher elastic seismic force demands. Hence, it was subsequently found that the columns would need to be strengthened.

The direct cost of implementing the above retrofit concept is significant. Furthermore, a substantial period of construction time is required to implement the above retrofit scheme. The indirect cost, resulting from the loss of production to allow for the retrofit construction work, is even higher.

SEISMIC ASSESSMENT AND RETROFIT BY THE DISPLACEMENT-BASED METHODOLOGY

Overview of the Displacement-Based Methodology

In contrast to satisfying design rules, the displacement-based methodology adopts a conceptually different approach. It attempts to simulate the true seismic response, particularly the non-linear response, by employing fundamental principles of mechanics and non-linear component behaviour using modern numerical simulation technologies. Rather than comparing elastic seismic force demands with component strength capacities, the displacement-based approach compares seismic deformation demands with deformation acceptance criteria established based on laboratory and full scale test data for various performance objectives (permitted degree of damage), on a component-by-component and element-by-element basis, to quantitatively determine the seismic performance (damage).

The displacement-based seismic assessment process begins with clearly, qualitatively and quantitatively defined seismic performance objectives, based on the usage of the structure and the requirements of the owner and those of legislation. The process continues with quantitative assessments of component strength and deformation capacities (acceptance criteria). Component seismic force and deformation demands are assessed by carrying out seismic response analyses. Often, non-linear seismic response analyses are carried out. The seismic force and deformation demands are then compared with their corresponding acceptance criteria on a component-by-component and element-by-element basis. Instead of hoping that the code ductile detailing provisions will provide sufficient ductility capacity, as implied in the force-based methodology, the displacement-based methodology actually assesses the ductility capacity and demand on a component-by-component basis. FEMA 273 [2] and ATC 40 [3] provide both qualitative and quantitative strength and deformation acceptance criteria established based on component laboratory test data.

Evaluation of Component Strength and Deformation Capacities and Acceptance Criteria

For force-controlled actions leading to brittle failure modes, such as compression, shear and torsion in reinforced concrete, the FEMA 273 acceptance criterion $\kappa Q_{CL} \geq Q_{UF}$ applies, in which Q_{CL} is the lower bound component strength, Q_{UF} is the design action due to gravity and earthquake loads obtained from non-linear analysis and κ is the knowledge factor.

The component strength capacities of the existing structure have been assessed and are summarised in Table 1. As pointed out previously, failure of the lower ring beams in combined shear and torsion is the most critical. This brittle failure mode prevents the development of the lower ring beams' full flexural strength and thus prevents the development of a ductile side-sway mechanism. As a result, the beams are classified as controlled by shear. Table 6-6 of FEMA 273 recommends that for such beams, the plastic hinge rotation capacity (deformation acceptance criteria) be taken as 0.0, implying no usable ductility capacity.

However, it is important to point out that had the shear strength been higher than the shear force demand determined based on the expected flexural strength, the beams would have possessed certain plastic deformation capacity. Had this been the case, flexural plastic hinge rotation at the ends of the lower ring beams would have been the critical mode of failure. According to Table 6-6 of FEMA 273, the acceptable plastic hinge rotation angle corresponding to the collapse prevention performance objective would have been 0.02 radians. After applying the knowledge factor, the values would have been reduced to 0.015 rad. This observation is crucial in leading to the displacement-based retrofit concept, as discussed in detail later.

The shear strength of columns is sufficient to allow the columns to develop their full flexural strength and hence form plastic hinges at both ends. Therefore, the columns are classified as controlled by flexure (deformation controlled). The applied axial force in the columns is small. According to FEMA 273 [3], the column plastic hinge rotation capacity (deformation acceptance criteria) is determined as 0.01 radians. After applying the knowledge factor κ=0.75, the design value of the column plastic hinge rotation angle capacity is 0.0075 radians.

Since joint shear failure is brittle in nature, beam-column joints are treated as force-controlled without any ductility capacity. The shear force demand on a joint is determined based on development of flexural plastic hinges in the beams framing into the joints. On this basis, the calculated joint shear force demand exceeds the joint shear strength capacity by a factor of approximately 2.0 as shown in Table 1. This implies that the joints are likely to fail in shear before flexural plastic hinges form in the adjacent beams.

Displacement-based Retrofit Concept and costs

A retrofit scheme alternative to the one led by the force-based approach is to transform the brittle shear failure of the ring beams and the beam-column joints to a ductile side-sway mode with ductile flexural plastic hinges developing and rotating at the beam ends. The seismic plastic hinge rotation demand under the MCE can then be assessed by a non-linear seismic response analysis procedure. Retrofit measures, if necessary, may be introduced to ensure the beam plastic hinge rotation capacity is higher than the seismic plastic hinge rotation demand.

As discussed previously, the plastic hinge rotation capacity of the ring beams in the existing structure is unusable because the shear strength capacity of these beams and joints is lower than the shear force demand. Hence, the key to unlocking the potential beam plastic deformation capacity is to strengthen the lower ring beams and the joints in shear without increasing the flexural stiffness and strength of these beams significantly. This can be achieved by employing a new technology, the carbon-fibre strengthening of reinforced concrete members. Carbon-fibre reinforcement sheets have a much higher tensile strength than normal steel reinforcement and can be bonded to the beam and joint faces, forming a composite section. This technology can substantially increase the beam and joint shear strength without significant increase in the beam flexural stiffness and strength. This is crucial for achieving the desired strength hierarchy and for avoiding an overall increase in the seismic force demand that conventional strengthening techniques would cause. Therefore, in this respect, the use of carbon-fibre strengthening is superior to the conventional retrofit technology of using steel plates. In addition to the above technical advantage, the retrofit employing carbon-fibre has other desirable advantages as well. The carbon-fibre material is light, durable, thin, corrosion resistant, easy to install and is maintenance free. The low quantity of material required to implement this retrofit scheme implies a low cost of material. Most importantly, due to its ease of installation, bonding of the carbon-fibre material does not require a major construction operation. The retrofit work can be carried out with only a few workers and a minimum requirement of equipment, as shown in Figure 3. The whole operation can be completed in a few night shifts, thus maintaining the continuation of normal production operations of the facility. No additional indirect costs would be incurred.

Figure 3 Carbon-fibre sheet material and its installation

Assessment of the Seismic Performance of the Retrofitted Structure by the Dispalcement-based Methodology

Once the objective of capacity design is achieved, namely achieving a desired strength hierarchy, the plastic deformation capacity in the lower ring beams becomes usable. The non-linear dynamic procedure can then be employed to evaluate the seismic deformation and force demands on a component-by-component basis and check if these are within the corresponding acceptance criteria (capacities).

After retrofit, the shear-torsion strength capacity of the lower ring beams and the beam-column joints at level +7.6 m becomes more than sufficient. As a result, the characteristics of the lower ring beams have been transformed, as summarised in Table 2. A comparison of deformation acceptance criteria of the existing and the post-retrofit structure, after applying the knowledge factor $\kappa = 0.75$, is summarised in Table 3.

TABLE 2 PRE- AND POST-RETROFIT CHARACTERISTICS OF THE LOWER RING BEAMS

Characteristic	**Existing Structure**	**Post-Retrofit**
Failure Mode	Shear	Flexure
Classification of Shear Reinforcement	Non-Conforming	Conforming
Acceptance Criteria	Force-Controlled	Deformation-Controlled

TABLE 3 SUMMARY OF DEFORMATION ACCEPTANCE CRITERIA OF THE EXISTING AND POST-RETROFIT STRUCTURE

Component	Acceptable Plastic Hinge Rotation (rad.)	
	Existing	Post-Retrofit
Lower Ring Beams	0.00	0.015
Upper Ring beams	0.00	0.00
Columns	0.0075	0.0075
Beam-Column Joints at +7.6 m	0.00	0.00

Since the displacement-based methodology attempts to simulate the actual behaviour of structures under the design earthquake, cracked component initial stiffness is adopted in the non-linear dynamic seismic response analysis. In accordance with recommendations of FEMA 273 and ATC 40, the cracked stiffness of beams and columns are assumed to be 50% and 70% of the corresponding un-cracked stiffness respectively.

In the non-linear dynamic analysis of the seismic response of the structure to the design earthquake, the earthquake input is represented by ground acceleration time histories. Three sets of tri-axial statistically independent MCE design spectrum compatible artificial earthquake records are used. The most unfavourable seismic force and deformation demands are used for the purpose of performance evaluation.

Table 4 summarises the primary results of the non-linear dynamic earthquake response analysis and comparisons with the deformation and force acceptance criteria where applicable. It can be seen that all seismic force and deformation demands are within the acceptable limits, indicating that the performance of the retrofitted structure under the MCE is satisfactory.

Figure 4 shows a contour plot of the maximum plastic hinge rotation angles of beams and columns. It can be seen that no plastic hinges have formed in the columns. The maximum flexural plastic hinge rotation in the lower ring beams is 0.0014 radians, being small compared with the acceptance criteria of 0.015 radians.

TABLE 4 SUMMARY OF NON-LINEAR DYNAMIC ANALYSIS RESULTS

Response Parameter	Maximum Value of Response
Shear-Torsion Strength Utilisation	
Upper Ring Beam	< 1.0 Acceptable
Lower Ring Beam	< 1.0 Acceptable
Column	<1.0 Acceptable
Flexural Plastic Hinge Rotation	
Upper Ring Beam	No flexural plastic hinge formed, Acceptable
Lower Ring Beam	= 0.0014 rad. < Acceptable limit 0.015 rad.
Column	No flexural plastic hinge formed, Acceptable
Joint Shear Strength Utilisation	
Beam-Column Joints at +17.3 m	< 1.0 Acceptable
Beam-Column Joints at +7.6 m	<1.0 Acceptable

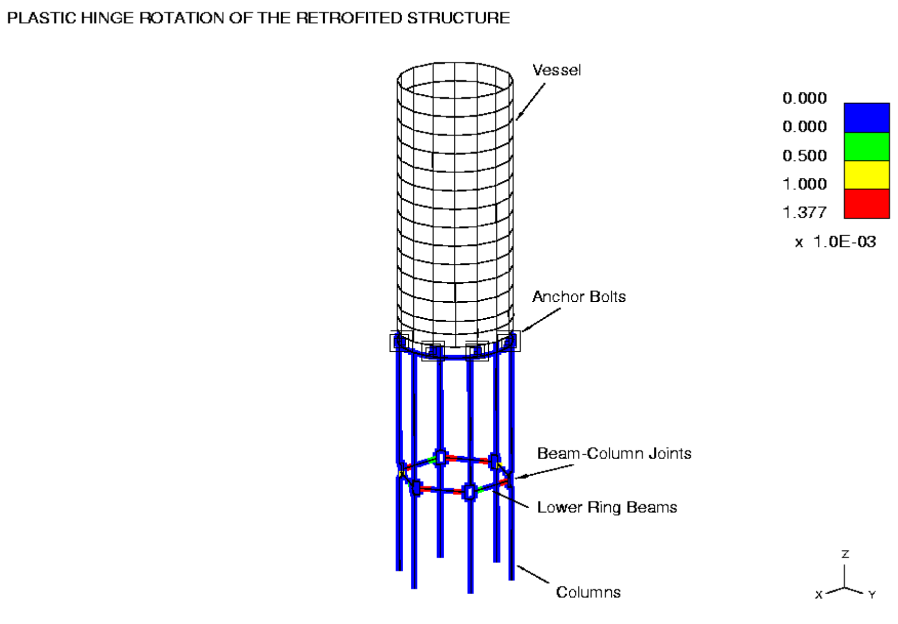

Figure 4: Maximum plastic hinge rotation angle of the retrofitted structure

DISCISSIONS AND CONCLUSIONS

The new displacement-based seismic design and assessment methodology differs fundamentally from the conventional force-based methodology. While the latter aims to satisfy design rules in the form of seismic design code provisions based on global force reduction factors, the former emphasises the understanding of the behaviour and the quantification of the performance (damage) of the structure under the design earthquake, based on examination of local individual element capacities and demands.

The conventional force-based method has a number of shortcomings particularly when applied for seismic assessment and rehabilitation of existing non-ductile structures. Firstly, force-based parameters, such as global force reduction factors, do not quantify damage directly, either structural or non-structural. Secondly, difficulties arise in determining an appropriate single force reduction factor for a structure that does not satisfy modern code ductile detailing provisions. While satisfaction of these provisions ensures adequate ductility capacity, non satisfaction of these provisions does not necessarily imply no ductility capacity. The limited ductility capacity inherent in components not satisfying code ductile detailing requirements cannot be utilised within the framework of the force-based methodology. The difficulty encountered in assigning appropriate force reduction factors for non-ductile structure cannot be overcome within the framework of the force-based methodology, since the force reduction factor is not a physical quantity that can be measured from laboratory component tests. This issue has to be resolved within the framework of the displacement-based methodology.

The newly emerging displacement-based methodology has a number of advantages, in particular when applied to assessment and rehabilitation of existing non-ductile structures. Firstly, displacement-based parameters better quantify structural and non-structural damage than force-based parameters. Secondly, the difficulty of establishing an appropriate single force reduction factor for a non-ductile structure is overcome by quantifying the limited ductility capacity inherent in these structures using displacement-based parameters on an element-by-element basis. The displacement-based parameters are physical quantities that can be measured directly in component laboratory tests. This is the approach adopted by FEMA 273 and ATC 40 to address seismic rehabilitation of existing non-ductile structures. Thirdly, the displacement-based method leads to better understanding of the behaviour and better quantification of performance of structures under the design earthquake and tends to lead to more effective and cost efficient rehabilitation schemes. This advantage is clearly demonstrated in this paper by the assessment and rehabilitation of an existing reinforced concrete structure.

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