Engineering Structures 43 (2012) 105–119

Contents lists available at SciVerse ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Performance limits for structural walls: An analytical perspective

İlker Kazaz [a], Polat Gülkan [b,*], Ahmet Yakut [c]

[a] Department of Civil Engineering, Atatürk University, 25240 Erzurum, Turkey
[b] Department of Civil Engineering, Çankaya University, 06810 Ankara, Turkey
[c] Department of Civil Engineering, Middle East Technical University, 06800 Ankara, Turkey

ARTICLE INFO

Article history:
Received 4 July 2011
Revised 13 May 2012
Accepted 14 May 2012
Available online 18 June 2012

Keywords:
Damage limits
Seismic codes
Plastic rotation
Shear walls
ASCE/SEI 41
Eurocode 8
Turkish Seismic Code

ABSTRACT

Recently proposed changes to modeling and acceptance criteria in seismic regulations for both flexure and shear dominated reinforced concrete structural walls suggest that a comprehensive examination is required for improved limit state definitions and their corresponding values. This study utilizes nonlinear finite element analysis to investigate the deformation measures defined in terms of plastic rotations and local concrete and steel strains at the extreme fiber of rectangular structural walls. Response of finite elements models were calculated by pushover analysis. We compare requirements in ASCE/SEI 41, Eurocode 8 (EC8-3) and the Turkish Seismic Code (TSC-07). It is concluded that the performance limits must be refined by introducing additional parameters. ASCE/SEI 41 limits are observed to be the most accurate yielding conservative results at all levels except low axial load levels. It is shown that neither EC8-3 nor TSC-07 specifies consistent deformation limits. TSC-07 suggests unconservative limits at all performance levels, and it appears to fall short of capturing the variation reflected in the calculated values. Likewise EC8-3 seems to fail to represent the variation in plastic rotation in contrast to several parameters employed in the calculation. More accurate plastic rotation limits are proposed.

1. Introduction

One of the most important steps of performance based assessment of RC buildings relies on comparison of deformations obtained from nonlinear structural analyses (static or dynamic) with the performance based limits. These deformation limits significantly affect the assessment result so their accuracy plays a critical role. Provisions for performance assessment of reinforced concrete structures, such as FEMA356 [1], Eurocode 8 [2] and ASCE/SEI 41 [3] include deformation limits for both flexure and shear controlled wall members at specific limit states to estimate the performance of components and structures. The criteria are defined in terms of plastic hinge rotations and total drift ratios for the governing behavior modes of flexure (ductile members) and shear (brittle members), respectively. Recently, strain limits are defined for concrete in compression and steel in tension at serviceability and damage-control limit states as a vital component of direct displacement-based design procedures [4]. The recently revised Turkish Seismic Code (TSC-07) [5] specifies limiting strain values associated with different performance levels of reinforced concrete members. While deformations are specified in relation to global parameters, local damage indicators in terms of strain limits are used inconsistently to determine the expected performance. For results of nonlinear pushover analyses to be evaluated according to either of the acceptance criteria, i.e. whether local or global response will imply similar performance states is a matter that must be established because full calibration of the requirements is lacking.

Another criticism raised against the rotations associated with different limit states is that they may turn out to be lower than the actual rotations expected to develop in reinforced concrete sections [6]. So, it is postulated that the given limits may be unduly conservative. In a way this is a direct consequence of adaptations performed for the plastic hinge analysis method employed in the guidelines. In most applications the moment–curvature relation of a section is calculated using the plane section assumption, the limiting plastic rotations in codes were adjusted to conform to the resulting plastic rotations calculated by multiplying the assumed plastic hinge length and plastic curvature rather than the actual rotations. These issues will be examined in this study to test adequacy of the limits specified by codes and guidelines. In previous studies [7,8] analytical modeling techniques for shear wall elements both in micro- and macro-levels were investigated and closed-form equations for estimation of building collapse capacity of moment resisting frame and shear wall structural systems subjected to seismic excitations were developed. We employ nonlinear finite element analysis for reinforced concrete structural components that has been thoroughly verified by benchmark problems as can be found in Kazaz [9].

* Corresponding author. Tel.: +90 312 233 1403.
E-mail address: polatgulkan@cankaya.edu.tr (P. Gülkan).

0141-0296/$ - see front matter
http://dx.doi.org/10.1016/j.engstruct.2012.05.011

Structural performance assessment, done either by the family of methods grouped under nonlinear static procedures (NSPs) or by using the more reliable nonlinear dynamic analysis needs to provide insight into how severe individual members are deformed. The bridge that gaps the global deformations and member end deformations must be reliable and stable. In this article we develop a tool that has been derived for walls that will provide improved estimates for member level deformations. Current modeling and acceptance criteria available in the provisions and codes will be summarized in the next section.

2. Code performance limits

A performance level describes a limiting damage condition that may be considered likely to be brought into existence for a given building under seismically induced deformations. Building seismic performance level is determined on the basis of structural member damage states. Basically all deformation-based provisions employ similar damage state definitions for reinforced concrete members. Structural members are classified as "ductile" and "brittle" with respect to their mode of failure in determining the damage limits. Fig. 1 shows the conceptualized force versus deformation curve used in ASCE/SEI 41 [3], TSC-07 [5] and EC8-3 [2] to specify member modeling and acceptance criteria for deformation-controlled actions. Three discrete Component Performance Levels and two intermediate Component Performance Ranges are defined in Fig. 1 to identify the performance level of a member. The terminology used in reference to damage states of a member differs among the documents. In ASCE/SEI 41 [3], the discrete Performance Levels are Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The intermediate Structural Performance Ranges are designated as Damage Control Range and the Limited Safety Range. In EC8-3 [2] the discrete Limit States are named as Damage Limitation (DL), Significant Damage (SD), and Near Collapse (NC). In TSC-07 [5] Damage Limits are Minimum Damage Limit (MD), Safety Limit (SL) and Collapse Limit (CL) (Fig. 1).

As indicated in Fig. 1, at the Collapse Prevention level (CP) member deformation capacities are taken at ultimate strength or at lateral displacement demand at which capacity begins to rapidly degrade for primary components. At the Life Safety level (LS), member deformation capacities are reduced by a (safety) factor of 4/3 over those applying at Collapse Prevention. For the Immediate Occupancy (IO) two definitions arise in reference to Fig. 1. While ASCE/SEI 41 and TSC-07 anticipate some degree of nonlinear deformation beyond the global yield for the immediate occupancy level and minimum damage, respectively, EC8-3 adopts the global yield point as the limit state for the damage limitation on the member.

Seismic assessment provisions investigated in this study establish the damage states on semantically similar definitions as stated

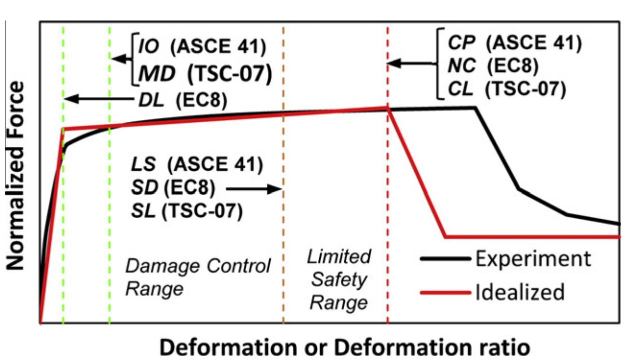

Fig. 1. Component performance levels.

above, but the differences between the deformation measures (drift, rotation, curvature and strain) and limiting values used as modeling and acceptance criteria for structural members can lead to significant differences in the estimations of global structural performance. For instance, while ASCE/SEI 41 and EC8-3 uses plastic rotation as the primary deformation parameter, TSC-07 uses section strains to assess the performance level of a member. Considering that these measures are interchangeable, the need for consistent limit values in different deformation measures is apparent.

2.1. ASCE/SEI 41 performance limits

ASCE/SEI 41 [3] basically adopts the same performance limits proposed in the wall provisions of FEMA 356 [1] for the seismic assessment and rehabilitation of existing buildings. According to ASCE/SEI 41 shear walls shall be considered slender if their aspect ratio is $H_w/L_w > 3.0$, and shall be considered short or squat if their aspect ratio is $H_w/L_w < 1.5$. Slender shear walls are normally controlled by flexural behavior; short walls are normally controlled by shear. The response of walls with intermediate aspect ratios is influenced by both flexure and shear. For walls deforming inelastically under lateral loading governed by flexure, the rotation (θ) over the plastic hinging region at the base of member will be used. For shear walls whose inelastic response is controlled by shear, the deformation limits are expressed in terms of the lateral drift ratios. For multi-story shear walls the drift shall be the story drift.

Table 1 gives the ASCE/SEI 41 plastic rotation limits for members controlled by flexure where P/P_o is the axial load ratio and v is the maximum average shear stress in the member normalized with respect to concrete compressive strength $\sqrt{f_c}$ calculated as

$$v = \frac{V_{\max}}{t_w L_w \sqrt{f_c}} \tag{1}$$

Here $V_{\max}$ is the maximum shear force carried by the member. The knowledge inherited in normalized shear stress expression given in Eq. (1) covers the parameters that affect the wall response significantly, so normalized shear is a useful parameter that discriminates the distinct behavior modes of wall response. ASCE/SEI 41 adopts the ACI 318-02 [10] requirements for the definition of a confined boundary.

Elwood et al. [11] proposes further changes to acceptance and modeling criteria for walls controlled by both flexure and shear, in order to make them more consistent with experimental results. For flexural walls the limiting average shear stress in Table 1 was increased from $0.25\sqrt{f_c}$ to $0.33\sqrt{f_c}$ (MPa) to obtain a better match with experimental results. Linear interpolation between tabulated values is to be used if the member under analysis has conditions that are between the limits given in the tables.

2.2. Eurocode 8

The deformation capacity of beam-columns and walls is defined as the chord rotation θ, i.e., the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span ($L_v = M/V$ = moment/shear), i.e., the point of contra-flexure. The chord rotation is also equal to the element drift ratio, i.e., the deflection at the end of the shear span divided by the length. The state of damage in a member is defined in EC8-3 [2] by three Limit States:

2.2.1. Limit state of Near Collapse (NC)

The value of the total chord rotation capacity (elastic plus inelastic part) at ultimate θ_{um} of concrete members under cyclic loading may be calculated from the following expression: