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A review of code seismic demands for anchorage of nonstructural components

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ABSTRACT

Prior to the 1989 Loma Prieta and 1994 Northridge earthquakes, performance standards for nonstructural building components – architectural pieces, mechanical equipment, and other attachments to buildings with non-trivial mass – were not well documented. Following widespread damage to nonstructural components during these events, a great deal of emphasis was placed by the National Earthquake Hazard Reduction Program on the development of a detailed and rational basis for seismic design that has been ongoing ever since. Given the prevalence of concrete slabs as floor elements in buildings, special attention was paid to the attachment mechanisms of said components into concrete. Most recently in this process, ACI 318-11 introduced a vaguely documented "overstrength factor", Ω_0, required for the design of certain types of nonstructural anchorage that increased demand forces on non-ductile anchors by a factor of 2.5. This factor is intended to provide a factor of safety against brittle failure modes and encourage design using ductile anchors – a sentiment adopted from the prevalence of ductility as a desirable feature in the seismic protection of building structures. To many, this overstrength factor might appear as a corollary to the similarly labeled Ω_0 used in the design of building structures as they are applied via the same load factor equation, but this is only superficially true. This paper clarifies such misconceptions, provides a detailed background of this factor's creation, discusses its evolution over the last decade, and documents the current research being performed for the assessment of its numerical values.

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1. Introduction

Nonstructural building components can be classified as elements within a structure which are not considered in its seismic resistance or response, yet develop significant lateral force demands from the structure's time-history responses during an earthquake. Historically, these demands have been poorly documented and studied, and saw only cursory mention in design codes prior to the early 1990's. The 1994 National Earthquake Hazard Reduction Program (NEHRP) provisions were the first attempt to quantify performance standards [1], attempting to emulate and adapt the philosophy behind building structure

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design for nonstructural applications.

Bachman and Drake [2] detailed the logic behind the early nonstructural component demand equations at the time. Their intent was to address several key performance concerns, including component mass distribution, location within the structure, environmental hazard risk, and importance of operation post-earthquake. A unique aspect of the provisions was the requirement for the component to have a ductile, energy-absorbing mechanism via the anchorage of the component, as no direct consideration for anchorage response was included in the lateral force equation. The omission of the anchorage response was an understandable simplification, as it depends on many factors that are not easily quantifiable. As a consequence, for the most part anchorage demands evolved independently from component demands, and component-independent ductility standards for anchors are applied universally in seismic codes.

One of the most significant challenges for the provisions was (and continues to be) the definition of ductility in the context of anchorage design. The approach adopted was to encourage ductile anchorage by applying penalty factors to “non-ductile” anchorage systems. The first attempts to quantify requirements for non-ductile anchors emerged in the 2003 NEHRP provisions [3], whereby the lateral force applied to the component is increased by a factor of 2.5. This factor was subsequently adopted in the 2006 IBC provisions. In 2008, ACI 318 implemented this factor as a 40% penalty on anchor capacity (50% for certain redundant anchorages). ASCE 7-05 required a multiplier of 130% on anchor forces and an additional multiplier on the horizontal earthquake force dependent on anchor ductility capacity, though due to errors in the text this second requirement was widely disregarded. ASCE 7-10 defers to ACI 318-11 for the design of anchorages for nonstructural components (in concrete) whereby ACI 318-11 D.3.3.4.3 (d) requires the application of Ω_0 for the design of anchors that do not satisfy one of requirements D.3.3.4.3 (a), (b), or (c). The values of Ω_0 for nonstructural components are given in ASCE 7-10 Supplement 1 (alternately, in ASCE 7-10 3rd printing). This is discussed further in the following:

In ACI 318-11, demand forces on ductile anchors can be directly resolved using the component's design force, provided the designer is able to sufficiently justify that the chosen anchors are indeed ductile. Brittle or non-ductile anchors, however, require that anchor demand forces be resolved using a multiplier of 2.5 on the component demand force. This scalar value is defined as an overstrength factor, Ω_0, and is intended to ensure linear-elastic behavior of brittle anchors, preventing potential premature failures – much like the overstrength factor Ω_0 for building structures is used to capacity-protect elastic members. Unlike earlier multipliers, the overstrength factor is applied only to the horizontal force in the demand equation and not as a reduction to the allowable anchor force. While seemingly innocuous, this shift changes the calculation of anchor forces for nonstructural components in a fundamental way. The application of multipliers to the calculated anchor force essentially increases the dead weight of the component along with the horizontal and vertical seismic components. Overstrength as defined in ASCE 7-10 is applied only to the horizontal seismic component, which increases the effective calculated anchor force for floor-mounted equipment (weight of the component and anchor force vectors oppose) and decreases it for hung components (weight of the component and the anchor force vectors coincide). This paper synthesizes the essential elements of the discussions that have led to these provisions.

2. The component demand force equation

As the overstrength factor effectively amplifies the component demand force equation, it is therefore of some benefit to explore the equation's history as it relates to nonstructural component anchorage. An early version of the current component design force equation, shown below in Eq. (1), appeared in the 1994 NEHRP provisions, and was used for the resolution of anchor demand forces.

$$F_P = \frac{a_p I_p W_p}{R_p}\left[C_a + (A_r - C_a)\left(\frac{x}{h}\right)\right] \tag{1}$$

R_p is a response modifier (based on the degree of component ductility capacity) that parallels the R-factor used for buildings, a_p is the component amplification factor to account for dynamic amplification (resonance) of the input (floor) motions associated with the component dynamic response; I_p is the importance factor of the component; W_p is the weight of the component; C_a is the soil-modified ground acceleration; A_r is the building roof acceleration; and x/h is the ratio of the floor on which the component is mounted to the total height of the structure. A value of either 1.0 or 2.5 was assigned for a_p, with 2.5 being the maximum amplification assumed for flexible components. Components with a natural period of less than 0.06 s are generally taken to be rigid, and thus the accelerations for the component match those of input floor motion ($a_p = 1.0$). It is interesting to note that these cutoff values are typically derived from the natural period of the component excluding anchorage response.

The rationale for the rigid/flexible paradigm arose from early research [2], derived from a comparative analysis of the natural period of the structure to the natural period of the component. Below 0.06 s, the spectral displacement plot for the component is relatively flat, and spectral accelerations are close to the peak input acceleration. Recent work by Fathali and Lizundia [4] has suggested that this assumption should be revisited, although that work does not discuss the potential influence of component anchorage response. The bracketed (height-dependent) portion of this equation is also of some interest, as it assumes a predominantly first-mode linear-elastic response of the building structure to which the component is attached. This was based off early studies [2] that indicated significant reductions in structural accelerations are unwarranted even if the building experiences significant nonlinear deformation.

The formulation discussed above was modified for the 1997 NEHRP provisions, and has remained unchanged to the present (see Eq. (2)). The C_a and A_r terms are replaced by the term $0.4S_{DS}$, where S_{DS} is the site-specific short period spectral acceleration.

$$F_P = \frac{0.4 a_p S_{DS} W_p}{\frac{R_p}{I_p}}\left[1 + 2\left(\frac{z}{h}\right)\right] \tag{2}$$

Upper and lower bounds (Eqs. (3) and (4), respectively) are also provided with this equation, as well as a seldom-employed formulation based on modal analysis (Eq. (5)).[1] This alternative allows the designer to directly find floor acceleration demand of the component, a_i, assuming potential building torsional amplification is accounted for by A_x. Though the determination of the a_i term requires the designer to compute the component's natural period while considering anchorage stiffness, this is purely for the component's elastic response and does not explicitly capture behavior where the anchorage system responds inelastically.

$$F_{P,min} = 0.3 S_{DS} I_p W_p \tag{3}$$

$$F_{P,max} = 1.6 S_{DS} I_p W_p \tag{4}$$

[1] Eqs. (2)–(5) are given in ASCE/SEI 7-10 as Equations 13.3.1 through 13.3.4, respectively.