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A method for determining the behaviour factor of moment-resisting steel frames with semi-rigid connections

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Abstract

In this paper, firstly, existing methods for determining the behaviour factor of moment-resisting steel frames and their range of applicability for multi-degree-of-freedom frames are reviewed. Modifications of this factor for multi-degree-of-freedom moment-resisting steel frames are then indicated. Necessary modifications in determining the behaviour factor of frames involve the period and the base shear distribution factor for a given earthquake loading code. The effects of storeys, spans and connections of frames on the behaviour factor were considered over ranges of values for these factors for frames with semi-rigid connections up to five spans and ten storeys. On the basis of the results, new relationships for the period and the behaviour factor of moment-resisting steel frames with semi-rigid connections as a function of the main geometric parameters of the frame are presented.

Keywords: Non-linear analysis; Dynamic analysis; Moment-resisting steel frames; Behaviour factor; Semi-rigid connection

1. Introduction

During the occurrence of an earthquake or similar extreme event it is possible for steel moment-resisting frames to enter a region of non-linear behaviour. The theoretical analysis of such behaviour is known to be a computationally demanding process and particularly for design it has been common to employ linear elastic analyses with reduced earthquake loading. Existing design codes deal with this through extracting the design force from spectra based on linear behaviour together with the use of a ‘behaviour factor’ that modifies the ‘linear’ force system to an equivalent one to account approximately for the non-linear effects. As will be described, there are two main methods for defining the behaviour factor. Experience has shown that these approaches have good accuracy for single-degree-of-freedom frames, but, because of using various simplifying hypotheses, for multi-degree-of-freedom frames the accuracy tends to be poor.

In this paper a new method for defining the behaviour factor is proposed, particularly for application to moment-resisting steel frames with semi-rigid connections. The proposed method uses a general definition of the behaviour factor and also considers loading effects as defined in earthquake loading codes.

2. Review of behaviour factor methods

The expected load–deflection (response) behaviour of a moment-resisting frame (MRF) with a single degree of freedom (SDOF) under static loading is shown in Fig. 1. Also shown is the manner in which that behaviour is commonly idealized. In Fig. 1 the deflections δ_s, δ_y and $\delta_{\max}$ represent, respectively, the displacements of the frame at the formation of the first plastic hinge, the deflection corresponding to the attainment of the plastic capacity of the idealized frame and the maximum displacement of the frame due to the applied loading. These allow the ductility factor μ to be defined as

$$\mu = \frac{\delta_{\max}}{\delta_y}. \quad (1)$$

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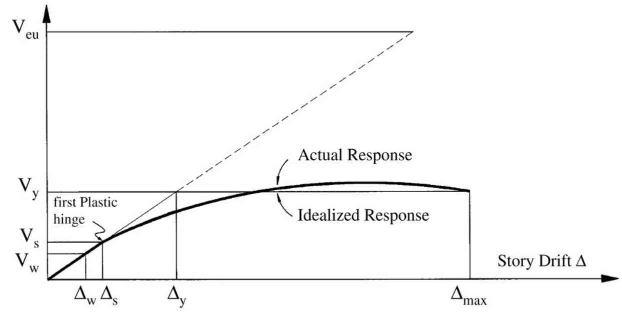

Fig. 1. Inelastic response of a SDOF moment-resisting frame.

The deflections of the frame are the results of applied loadings, most effectively defined in terms of the base shear V. In terms of base shear forces the behaviour factor R may be defined as [1,2] (American approach)

$$R = \frac{V_{eu}}{V_w} = \frac{V_{eu}}{V_y} . \frac{V_y}{V_s} \frac{V_s}{V_w} = R_\mu . R_\Omega . R_w \tag{2}$$

where V_w is the actual base shear set up by the applied dynamic loading (earthquake) and V_{eu} is the equivalent base shear required to represent the load effect for elastic analysis purposes. As shown in (2), the behaviour factor may be decomposed into partial factors defined as follows. The ductility reduction factor R_μ is the ratio of the equivalent 'elastic' base shear to the base shear at first yielding of the frame. It indicates the capacity of the frame for energy absorption. R_Ω is the so-called 'overstrength' factor and is equal to ratio of the yield base shear to the base shear corresponding to the formation of the first plastic hinge in the frame. The third partial factor R_w is the allowable stress reduction factor for design and is equal to the ratio of the base shear corresponding to the formation of the first plastic hinge to that corresponding to the allowable stress state for the frame as defined by conventional design codes. When the design method used is based on ultimate strength design, this factor is equal to unity [1,2].

While the concept is simple enough, and the definition of first yielding of the frame is clear, the definitions of the other terms are less so. This includes the definition of the yielding of the frame and the associated collapse mechanisms, particularly for multi-degree-of-freedom frames. To determine these values it is necessary that simplifying assumptions be made.

For example, when the design method used is based on ultimate strength, the allowable stress reduction factor R_w is equal to unity. It is clear that this definition will be correct for a single-degree-of-freedom system and also if it is optimally designed. However, in practice this factor will be defined through code-specified earthquake loading forces, so the assumptions of the earthquake loading code are involved in its evaluation. These assumptions may have an important effect in connecting the earthquake dynamic forces and the corresponding code values and hence in determining the behaviour factor R [1,2].

Instead of the behaviour factor R, in the European approach to this problem there is a code-specified frame response factor

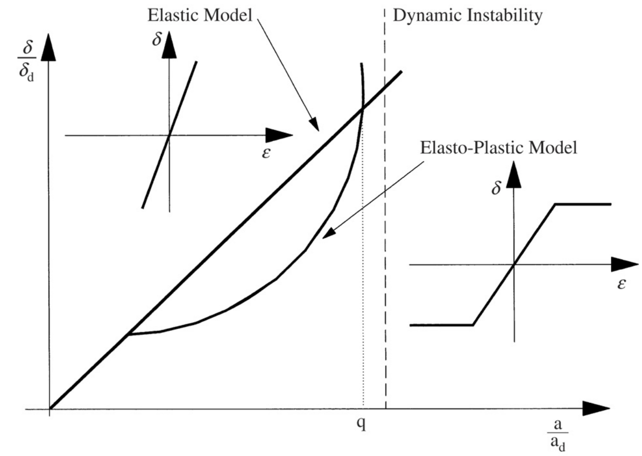

Fig. 2. Evaluating q on the basis of ductility factor theory.

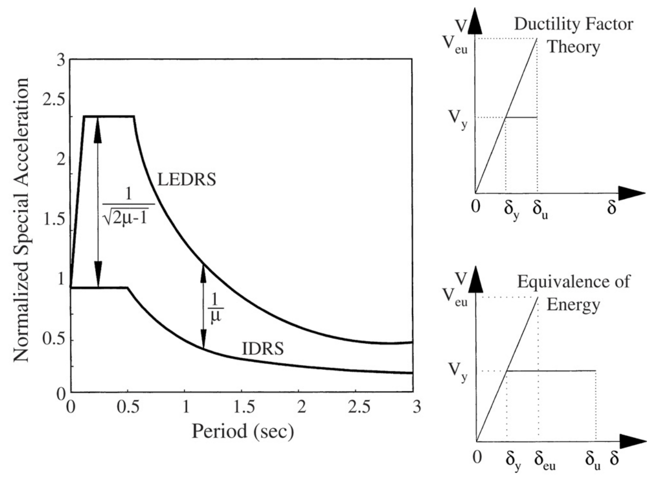

Fig. 3. Evaluating q on the basis of the response of SDOF systems.

q, defined as

$$q = \alpha \frac{A_u}{A_s} \tag{3}$$

where A_u is the peak ground acceleration leading to collapse and A_s is the peak ground acceleration corresponding to the first yielding of the frame. The factor α allows for different types (geometries) of frames.

Existing methods for evaluating A_u, A_s and hence q can be classified into three categories [3]: (i) those based on ductility factor theory; (ii) extensions from the results concerning the dynamic inelastic response of single-degree-of-freedom systems; and (iii) energy methods.

The first method can be illustrated with the aid of Fig. 2 [4,5]. For this method, q is derived from a series of dynamic inelastic analyses, for which the peak ground acceleration is increased step by step. At each step the ratios A/A_d and δ/δ_d are computed, where A_d and δ_d are respectively the design acceleration and the corresponding maximum displacement evaluated by means of a first-order elastic analysis. The design based on the elastic spectrum is valid until δ/δ_d is less than A/A_d; therefore the maximum value which can be assigned to