Chapter 3

Performance-Based Plastic Design (PBPD) Procedure

3.1 General

An outline of the step-by-step, Performance-Based Plastic Design (PBPD) procedure follows, with details to be discussed in subsequent sections in this chapter and theoretical justification given in the Appendix.

1. Select a desired yield mechanism and target drift for the structure consistent with the intended performance objectives for the design earthquake hazard. Assume idealized elastic-plastic (EP) force-displacement behavior and estimate the yield drift ratio, θ_y, for the structure.

2. Estimate the natural period, *T,* of the structure and assume an appropriate vertical distribution of design lateral forces.

3. With the information in steps 1 and 2 along with the design spectral acceleration value, S_a (Figure 1-1), calculate the design base shear, *V,* by equating the work needed to monotonically push the structure up to the target drift (no pushover analysis needed) to the energy needed by an equivalent EP-SDOF to be displaced up to the same drift. A rational theory of inelastic seismic response of EP-SDOF can be used here, such as the idealized inelastic response spectra by Newmark-Hall or others as preferred.

4. Modification for *V* is needed if the force-deformation behavior of the structure is different from the assumed EP behavior, such as for CBFs or other framing systems.

5. Use the plastic method to design the structural members that are expected to dissipate the earthquake energy inelastically (DYMs), while keeping the vertical distribution of lateral strength of the structure close to the distribution of design shear distribution. Members that are required to remain elastic (non-DYMs), such as columns, are designed by a capacity-design approach, by accounting for the strain-hardening and material overstrength of the DYMs as well as by including the frame deformation ($P - \Delta$) effects as appropriate.

3.2 Design Procedure

3.2.1 Target Yield Mechanism

Figure 3-1 shows several typical structural systems in the yield mechanism states subjected to design lateral forces and pushed to their target plastic drift limits. All inelastic deformations are intended to be confined within the DYMs that are part of the selected yield mechanism, such as plastic hinges in beams or yielding and buckling of bracing members. Since the plastic hinges at column or wall bases generally form during a major earthquake, the global yield mechanism of these structural systems also includes plastic hinges at those locations.

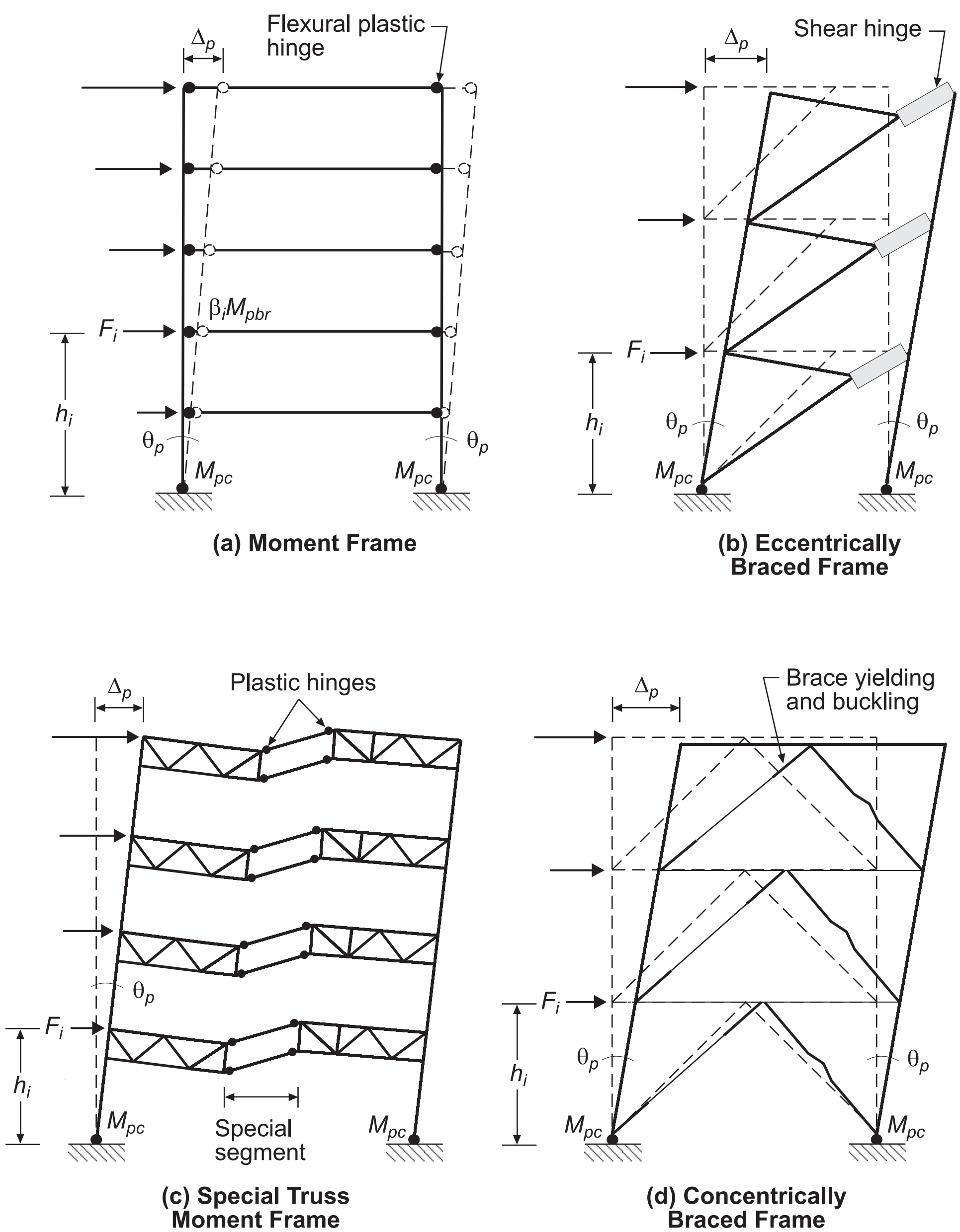
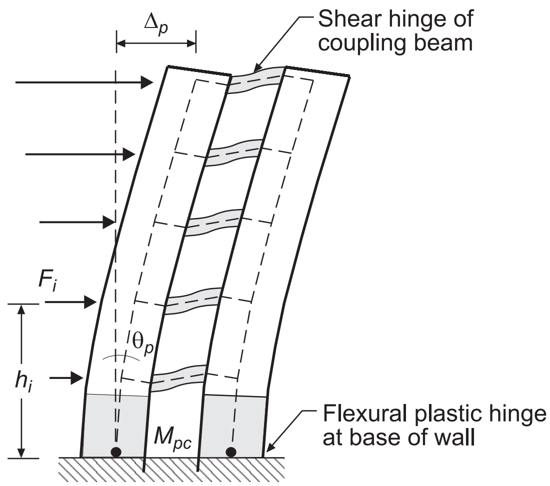


Figure 3-1
Desirable Yield Mechanisms of Typical Structural Systems



(e) Coupled Wall System

Figure 3-1 (continued)
Desirable Yield Mechanisms of Typical Structural Systems

3.2.2 Design Lateral Forces

Equivalent static design lateral forces in the current codes are obtained from simplified models assuming that the structures behave elastically and primarily in the first mode of vibration (ATC, 1978; Clough and Penzien, 1993; Chopra, 2000; BSSC, 2003b). However, building structures designed according to current code procedures are expected to undergo large deformations in the inelastic range when subjected to major earthquakes, thereby leading to lateral force distributions that can be quite different from those given by the code formulas. In order to achieve the main goal of performance-based seismic design, i.e., a desirable and predictable structural response, it is necessary to account for inelastic behavior of structures directly in the design process.

Unlike the force distribution in the current codes, the design lateral force distribution used in the PBPD method is based on maximum story shears as observed in nonlinear time-history analysis results (Chao et al., 2007). This new design lateral force distribution has been found suitable for MFs, EBFs, CBFs, and STMFs. Analytical results have shown that: 1) frames designed with this lateral force distribution experienced more uniform maximum interstory drifts along the height than the frames designed with current code distributions; 2) this force distribution also gives a very good estimate of maximum column moment demands when the structures are responding to severe ground motions and deform into the inelastic range; 3) higher mode effects are well reflected in the proposed design lateral force distribution. This lateral force distribution is expressed as

$$F_i = C'_{vi} V \tag{3-1}$$

where

$$C'_{vi} = (\beta_i - \beta_{i+1}) \left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j} \right)^{0.75 T^{-0.2}} \quad \text{when } i = n,\ \beta_{n+1} = 0 \tag{3-2}$$

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n} \right)^{0.75 T^{-0.2}} \tag{3-3}$$

In the above equations, β_i represents the shear distribution factor at level i; V_i and V_n, respectively, are the story shear forces at level i and at the top (nth) level; w_j is the seismic weight at level j; h_j is the height of level j from the base; w_n is the weight at the top level; h_n is the height of roof level from base; T is the fundamental period; F_i is the lateral force at level i; and V is the total design base shear.

3.2.3 Design Base Shear

The design base shear in the PBPD method is derived based on the inelastic state of the structure, with the drift control built in. Therefore, no separate drift check is needed after design. In this approach the design base shear is determined by pushing the structure monotonically up to a target drift after the formation of a pre-selected yield mechanism. No actual pushover analysis is needed for this, as will be seen later. The amount of work needed is assumed as a factor γ times the elastic input energy $E(=\frac{1}{2} MS_v^2)$ for an equivalent EP-SDOF system (Housner, 1956 and 1960). Housner (1960) used this approach in order to determine the collapse limit strength of a cantilever column (representing a water tower, for example). For simplicity, Housner assumed the energy factor $\gamma = 1$, as he did not have a good way of determining its value at that time. The above-mentioned work assumes no relationship with the actual energy dissipated during earthquake excitation, which has been used in energy-based procedures as proposed by a number of investigators (Akiyama, 1985; Uang and Bertero, 1988). However, those procedures have been found to be quite cumbersome to implement in common design practice. In the PBPD method, the needed work term $(E_e + E_p)$ is simply used as a means to calculate the required design base shear by establishing ties among the desired yield mechanism, design drift, force-displacement characteristics of the structure, and elastic input energy from the design ground motion. Thus, the work-energy equation can be written as

$$(E_e + E_p) = \gamma E = \gamma(\tfrac{1}{2} MS_v^2) \tag{3-4}$$

where E_e and E_p are, respectively, the elastic and plastic components of the energy (work) needed to push the structure up to the target drift. S_v is the design spectral pseudo-velocity, and M is the total mass of the system. The energy modification factor, γ, depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ). Figure 3-2 shows the relationship between the base shear (CW) and the corresponding drift (Δ) of the elastic and corresponding elastic-plastic SDOF systems.

Using the geometric relationship between the two areas representing work and energy in Figure 3-2, Equation (3-4) can be written as

$$\frac{1}{2}C_y W(2\Delta_{max} - \Delta_y) = \gamma\left(\frac{1}{2}C_{eu}W\Delta_{eu}\right) \tag{3-5}$$

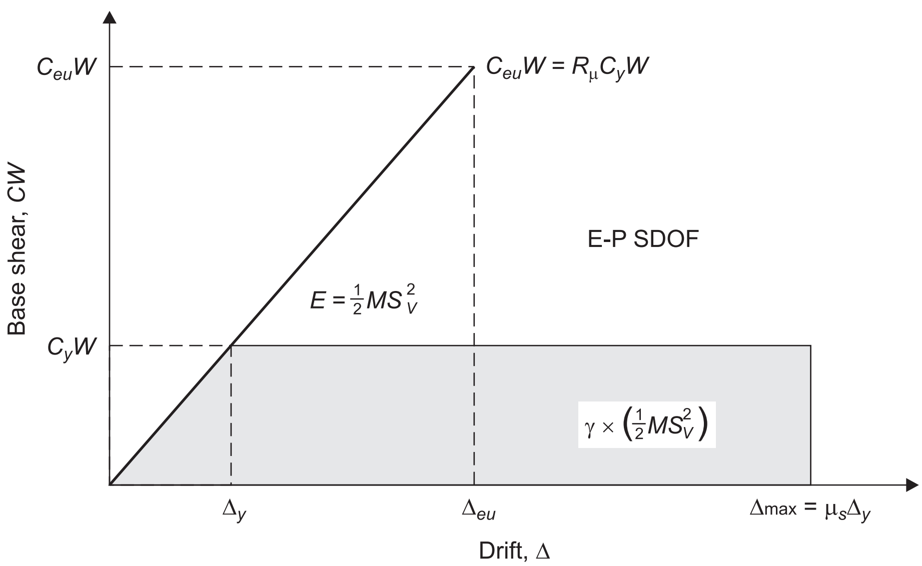


Figure 3-2
Structural Idealized Response and Energy (Work) Balance Concept for SDOF

Equation (3-5) can be reduced into the following form:

$$\gamma\frac{\Delta_{eu}}{\Delta_y} = \frac{(2\Delta_{max} - \Delta_y)}{\Delta_{eu}} \tag{3-6}$$

where Δ_{eu} and Δ_{max} in Figure 3-2 are equal to $R_\mu\Delta_y$ and $\mu_s\Delta_y$, respectively. Substituting these terms into Equation (3-6), the expression for energy modification factor γ can be written as

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{3-7}$$

where μ_s is the ductility factor equal to the design target drift divided by the yield drift (Δ_{max}/Δ_y), and R_μ is the ductility reduction factor equal to C_{eu}/C_y. It can be seen from Equation (3-7) that the energy modification factor γ is a function of the ductility reduction factor (R_μ) and the ductility factor (μ_s). The method by Newmark and Hall (1982) is used herein to relate the ductility reduction factor and the structural ductility factor for EP-SDOF as shown in Figure 3-3 and Table 3-1 (Miranda and Bertero, 1994; Lee and Goel, 2001). Plots of the energy modification factor γ as obtained from Equation (3-7) are shown in Figure 3-4. For this purpose, any inelastic spectra for EP-SDOF systems can be used as preferred.

Table 3-1
Ductility Reduction Factor ($R_\mu = C_{eu}/C_y$) and its Corresponding Structural Period Range

Period Range	Ductility Reduction Factor
$0 \le T < \frac{T_1}{10}$	$R_\mu = 1$
$\frac{T_1}{10} \le T < \frac{T_1}{4}$	$R_\mu = \sqrt{2\mu_s - 1} \cdot \left(\frac{T_1}{4T}\right)^{2.513 \cdot \log\left(\frac{1}{\sqrt{2\mu_s - 1}}\right)}$
$\frac{T_1}{4} \le T < T'_1$	$R_\mu = \sqrt{2\mu_s - 1}$
$T'_1 \le T < T_1$	$R_\mu = \frac{T\mu_s}{T_1}$
$T_1 \le T$	$R_\mu = \mu_s$

Note: $T_1 = 0.57$ sec., $T'_1 = T_1 \cdot (\sqrt{2\mu_s - 1}/\mu_s)$ sec.

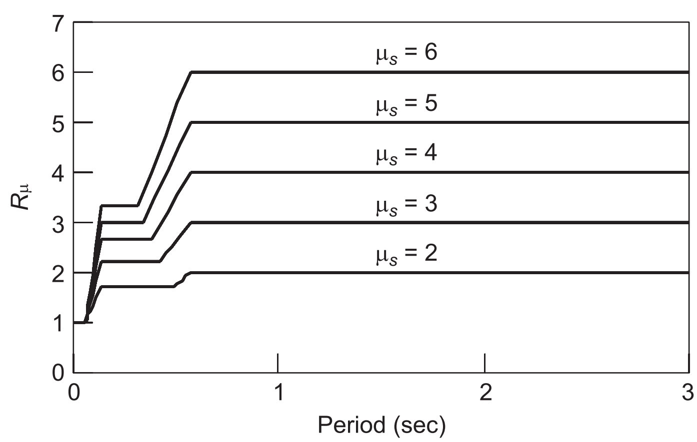


Figure 3-3
Idealized Inelastic Spectra by Newmark and Hall for EP-SDOF (1982)