Chapter 2
Alternative Approaches to the Seismic Analysis of R/C Bridges

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Abstract Two alternative approaches for the design of R/C bridges are compared in this work, namely the traditional Strength Based Design (SBD) and the Direct Displacement Based Design (DDBD). It is found that these two methods give the same results when the same set of assumptions are employed. These are (a) the yield curvature (displacement) is nearly invariant for the chosen type of steel and geometry of the critical cross-section, (b) the equivalent pre-yielding stiffness is strongly correlated to the strength, and (c) the equal displacement rule is applied in both cases. The basic assumptions and properties behind the non-linear pushover-based methods, which are included in modern design codes, are reviewed and some specifics related to their use for the analysis of bridges are presented and briefly discussed.

Keywords Seismic analysis · Strength based design · Direct displacement based design · Pushover based analysis · Bridges · Yield displacements · Pre-yield stiffness · Effective stiffness

2.1 Introduction

Although that the response of most R/C bridges, which are subjected to strong earthquake load is predominantly nonlinear, their design is typically based on the results of elastic methods of analysis. For example, the Eurocode 8/2 standard (CEN 2005) defines the modal response spectrum analysis as the basic method of analysis. The acceleration design spectrum, used in this type of analysis, is typically reduced based on the chosen behaviour (reduction) factor for the bridge at hand.

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This reduction defines the required strength of the structure. The larger reduction of forces, in general, means that the provided strength of the structure would be smaller. Taking into account the basic principle of the earthquake engineering, namely the equal displacement rule, further means that smaller provided strength should be accompanied by larger ductility capacity in the structure.

Recently, some doubts have been expressed about the validity of the equal displacement rule. A new design approach "Direct displacement based design"—DDBD was proposed (Priestley et al. 2007) as an alternative to the traditional "Strength Based Design"—SBD. Several opinions that DDBD is more economical than SBD have been presented (e.g., Martini 2007; Rahman and Sritharan 2011). In Sect. 2.2 a comparison of these methods is provided. Since both methods suppose that the response of structure is governed predominantly by one mode of vibration, they are compared considering only those structures, which can be modelled using single-degree-of-freedom (SDOF) model with reasonable accuracy. Comparison is limited to structures with fundamental periods of vibration in the constant velocity region of the spectrum.

In Sects. 2.2.1 and 2.2.2, a discussion about the basic assumptions of DDBD and the equal displacement rule is presented, respectively. The comparison of the methods is provided in Sect. 2.2.3 taking into account same basic assumptions, namely (a) the yield displacement is almost invariant for the chosen quality of the steel and the geometry of the structure, (b) the equivalent pre-yielding stiffness is strongly correlated to the strength, and (c) the equal displacement rule is applied in both cases. It means that both methods were modified as (a) in SBD the pre-yielding stiffness was estimated taking into account the basic assumption of DDBD method (the invariant yield displacement) and (b) in DDBD the equivalent damping was defined taking into account the basic assumption of SBD (the equal displacement rule). The modified methods are also compared with their original versions.

As previously mentioned, the seismic response of most RC bridges is non-linear. Thus the majority of the modern codes and design guidelines introduce the non-linear methods into design practice in order to estimate the seismic response more realistically. In general, the most refined and accurate inelastic method is the nonlinear response history analysis (NRHA). Nevertheless, it is only infrequently used in the design practice, since it is, for the moment, too complex for regular use by practising engineers. To simplify the nonlinear analysis and to make it more suitable for design practice, different static inelastic methods have been developed. Most of them are based on the pushover analysis. They are considered more user-friendly and relatively easy to understand. An overview of basic features of such methods, which are included in different codes, is provided in Sect. 2.3.

Most of the above methods have been primarily developed for the assessment of the seismic response of buildings. Since the response of bridges is often quite different from that of buildings, specific items that should be taken into account when these methods are applied to bridges are also briefly reviewed and discussed in Sect. 2.3.

2.2 Basic Concepts and Comparison of the DDBD and SBD

2.2.1 Basics of the DDBD

The DDBD is not a standard design tool. Therefore the main steps of the method are first reviewed (see "*The basic steps of DDBD*"). One of the basic assumptions of this method is that the yield curvature of critical cross-sections is almost invariant and depends only on the chosen quality of the reinforcing steel and chosen geometry of structural elements. This further implies that the yielding displacement does not depend on the strength of the structure, but only on its geometry. This assumption is discussed in sub-section "*Constant yield displacement*". The equal displacement rule is not included into DDBD. Since it is demonstrated in Sect. 2.2.3 that DDBD and SBD give the same results, when the same input data and the assumptions are used, the DDBD is modified at the end of this section (see *Modified DDBD taking into account equal displacement rule*) taking into account this basic principle of the seismic engineering.

The basic steps of the DDBD

The first step of DDBD is the estimation of the yield displacement Δ_y. It can be estimated in the way, described in the next sub-section. In the second step the ultimate displacement Δ_u is defined based on the yield displacement and the chosen ductility μ, or by taking into account drift limitations. If drift limitation is relevant, the ductility μ is calculated based on the ratio of the ultimate displacement (corresponding to drift limitation) and the yield displacement.

In the next step, the equivalent viscous damping at the peak response ξ_{eq} (corresponding to the ultimate displacement) is estimated, based on the type of the analysed structure and the type of used material. In the case of concrete bridges with effective periods longer than 1 s, the following equation has been proposed (Priestley et al. 2007):

$$\xi_{eq} = 0.05 + 0.444((\mu - 1)/\mu\pi) \tag{2.1}$$

Then the equivalent viscous damping is used to reduce the displacement spectrum, which corresponds to standard 5% damping as:

$$S_d(T)_{\xi_{eq}} = S_d(T)_{5\%}\sqrt{\frac{0.07}{0.02 + \xi_{eq}}} \tag{2.2}$$

Note that the displacement spectrum can be calculated based on the elastic acceleration spectrum taking into account the relationship:

$$S_d(T) = \frac{S_a(T) \cdot T^2}{4\pi^2} \tag{2.3}$$

Equation (2.2) can be written in a modified form as:

$$S_d(T)_{\xi_{eq}} = S_d(T)_{5\%} c_r \tag{2.4}$$

where the coefficient c_r is expressed as:

$$c_r = \sqrt{\frac{0.07}{0.07 + 0.444\left(\frac{\mu - 1}{\mu\pi}\right)}} \tag{2.5}$$

In the next step the maximum displacement Δ_u is used to estimate the corresponding period of vibration of the structure T_{sec} as it is illustrated in Fig. 2.1. Note that this period is related to the secant stiffness of the structure at the maximum displacement Δ_u.

Based on T_{sec} the corresponding stiffness k_{sec} is defined. The final step of the methods includes the estimation of the required strength of the structure. It is defined multiplying the k_{sec} by the maximum displacement Δ_u. Summarizing all the steps, the required strength of the structure can be calculated as a function of the maximum displacement Δ_u:

$$F_R = \frac{4\pi^2 m S_d^2(T_D)_{\xi_{eq}}}{T_D^2 \Delta_u} = \frac{4\pi^2 m c_r^2 S_d^2(T_D)_{5\%}}{T_D^2 \Delta_u} \tag{2.6}$$

where F_R is the required strength, m is the mass of the structure, c_r is the reduction factor depending on the ductility (see Eq. 2.5), Δ_u is the ultimate displacement, T_D is the corner period (at the end of the constant velocity region of the spectrum),

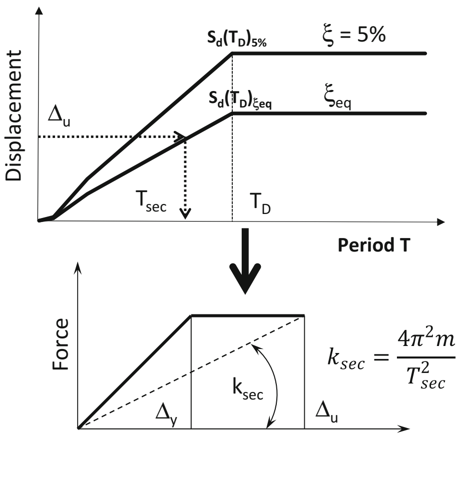

Fig. 2.1 Calculation of the effective period T_{sec} corresponding to ultimate displacement Δ_u

$S_d(T_D)_{\xi eq}$ and $S_d(T_D)_{5\%}$ are the corresponding spectral displacement at equivalent damping ξ_{eq} and 5% damping, respectively.

The constant yield displacement

One of the basic assumptions of the DDBD is that the yield displacement Δ_y is almost invariant once the quality of the reinforcement and the geometry of the structure are defined. For structures, which can be represented by SDOF models with reasonable accuracy, it can be estimated as:

$$\Delta_y = \frac{\phi_y (H + L_{sp})^2}{3} \tag{2.7}$$

where Δ_y is the yield displacement, H is the effective height (in cantilever columns with the mass concentrated at the top of the column, the effective height is equal to the height of the column), L_{sp} is the strain penetration length, and ϕ_y is the yield curvature. The yield curvature can be expressed as a function of the yield deformation of steel and the height of the cross-section. For example, in the case of circular RC concrete columns the following equation has been proposed:

$$\phi_y = \frac{2.25\varepsilon_y}{D} \tag{2.8}$$

where ε_y is the yield deformation of the steel ($\varepsilon_y = f_y / E_s$, where f_y and E_s is the yield stress and the modulus of elasticity of the steel, respectively), and D is the diameter of the cross-section. Similar expressions are proposed for other types of structural elements (columns, walls, beams, steel cross-sections), and frames (Priestley et al. 2007).

The previous observation is illustrated using the example of a cantilevered bridge column. In Fig. 2.2 the moment-curvature relationships for bridge column corresponding to different levels of the axial load is presented.

The actual moment-curvature relationship is idealized by means of a bilinear relationship, as is shown in Fig. 2.2b by a dashed line. The initial slope of this relationship defines the equivalent (effective) stiffness, which is typically defined as:

$$E_c I_{eq} = \frac{M_{y1}}{\phi_{y1}} \tag{2.9}$$

In the above, E_c is the modulus of elasticity of the concrete, I_{eq} is the equivalent (effective) moment of inertia of the cross-section, and M_{y1} (9830 kNm) and ϕ_{y1} (2.15 10^{-3} 1/m) are the bending moment and the curvature corresponding to the yielding of the first layer of flexural reinforcement, respectively (see point P1 in Fig. 2.2). If there is no strain hardening, the yield moment M_y can be taken to be equal to the flexural strength M_R. The yield curvature can be estimated as $\phi_y = M_R / M_y\ \phi_{y1}$. In the presented case this value coincides quite well with the value

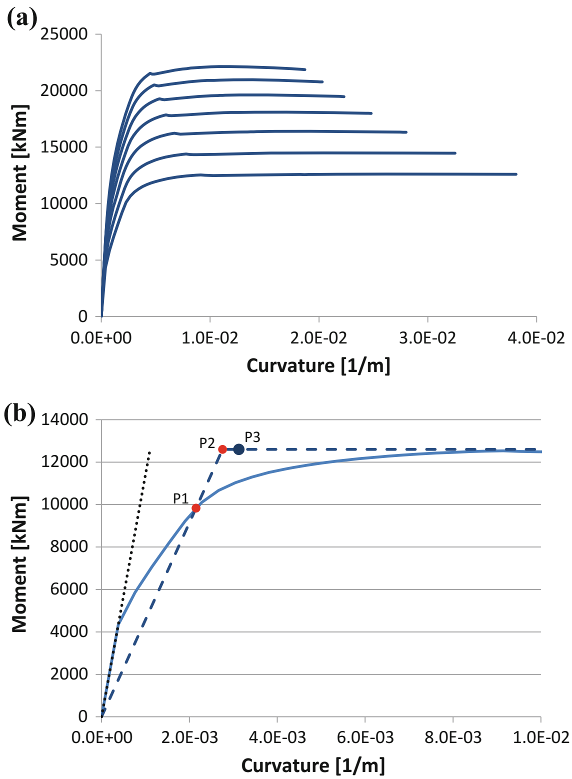

Fig. 2.2 The moment-curvature relationship of critical cross-section in bridge column: **a** different levels of the axial load, **b** idealization

estimated by using Eq. 2.8 (see point P3 in Fig. 2.2). More details about the estimation of the yield displacement can be found elsewhere (Priestley et al. 2007).

Modified DDBD taking into account the equal displacement rule

In the original DDBD the equal displacement rule is not applied. When this rule is taken into account, the relationship between the required strength and chosen ultimate displacement Δ_u can be expressed in the same way as in the original DDBD (see Eq. 2.6); however the coefficient c_r should be replaced by:

$$c_{r1} = \frac{1}{\sqrt{\mu}} \text{ and } c_{r2} = \frac{1}{\sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}}} \tag{2.10}$$

for cases with and without strain hardening, respectively (see Fig. 2.3). In these equations μ is the displacement ductility and α is the strain hardening.

The original and modified DDBD are compared in Sect. 2.2.3. It is demonstrated that in the majority of cases the original DDBD gives more conservative results than its modified version.

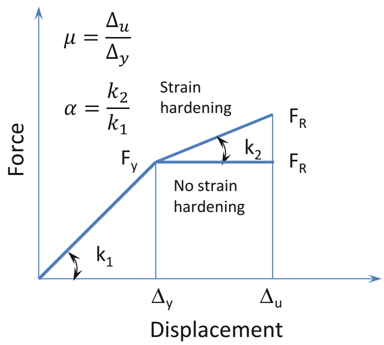

Fig. 2.3 Response with and without the presence of strain hardening

2.2.2 Basics of the SBD and the Equal Displacement Rule

Contrary to the DDBD, where the basic (or assigned) quantities are displacements and the ductility capacity of the structure, in SBD these quantities are the strength of the structure and the behaviour factor (reduction of seismic forces). Based on the strength of the structure, the ultimate displacements are typically estimated employing the equal displacement rule. The traditional interpretation of this rule (see Fig. 2.4a) can be however misleading. Thus some researchers expressed their doubts about its validity (Priestley et al. 2007). Therefore, the discussion about various interpretations of equal displacement rule is provided in sub-section *The equal displacement rule*.

Although the SBD is routinely used in the everyday design, some issues related to this type of design are discussed in sub-section *Basic equations of SBD, taking into account invariant yield displacement*. A special attention is devoted to estimation of the initial (pre-yielding) equivalent stiffness of the structure. It is defined taking into account the basic assumption of DDBD about the invariant yield displacement and taking into account the basic principle of earthquake engineering—the equal displacement rule.

The equal displacement rule

Extensive research has shown that, in the case of different systems with natural periods in the medium and long period range, the seismic demand, in terms of the displacements Δ, is independent of the strength of the system and is equal to the displacement demand Δ_e of an elastic system with the same natural period. This is the so called "equal displacement rule", which was defined by Veletsos and Newmark (1960), and has been used successfully for more than half of a century. The results of many statistical studies (e.g. Konakli and Der Kiureghian 2014) have confirmed the applicability of this rule to structures which are on firm sites with fundamental periods within the medium or long-period range, with relatively stable and full hysteretic loops.

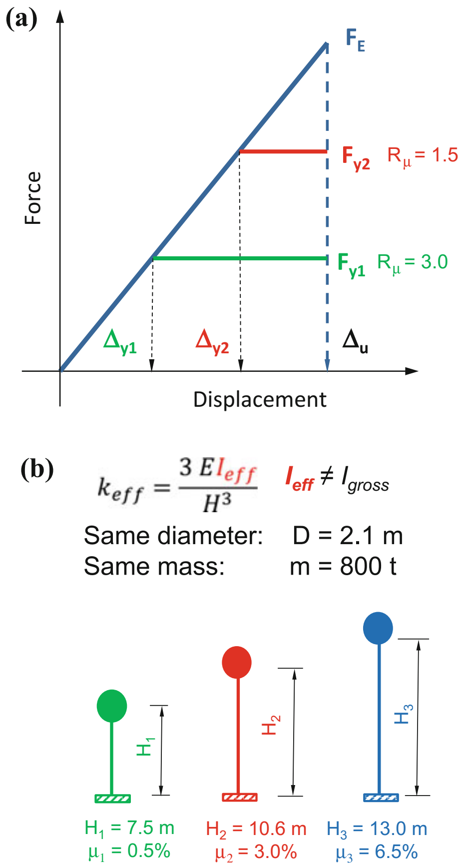

Fig. 2.4 The equal displacement rule: **a** traditional presentation, **b** an example of structures that are used to illustrate the equal displacement rule

The traditional interpretation of the equal displacement rule is presented in Fig. 2.4a. It is often interpreted as the response of one (the same) structure, where different strength levels (reduction of forces) are provided. However, this interpretation is not correct, since the larger reduction of the seismic forces typically means the larger reduction of the pre-yielding stiffness, which is not the case in the Fig. 2.4a, where the pre-yielding stiffness is the same. That is why some researchers expressed their doubts about the validity of this rule.

Actually, Fig. 2.4a presents the response of three different structures, which have the same initial (pre-yielding) period of vibration (same pre-yielding stiffness and the same mass) but different strengths.

Let's say that it represents the response of three cantilever columns, presented in Fig. 2.4b, which have the same mass and diameter, but their heights and longitudinal reinforcement are substantially different. Their strengths are inversely proportional to the chosen reduction of forces (1, 1, 5, 3 for tallest, medium and shortest column, respectively). Their initial effective stiffness k_{eff} can be defined as

$$k_{eff,i} = \frac{3EI_{eff,i}}{H_i^3} \tag{2.11}$$

where E is modulus of elasticity, $I_{eff,i}$ and H_i are the effective (pre-yielding) moment of inertia and the height of the i-th column, respectively.

The yielding force of the tallest column is 3 times larger than that of the shortest column. The height of the tallest column is 1.73 times larger than that of the shortest column. The pre-yielding stiffness $k_{eff,i}$ is the same. That means that the effective moment of inertia should be proportional to the height of the columns H_i^3.

$$\frac{I_{eff,3}}{I_{eff,1}} = \left(\frac{H_3}{H_1}\right)^3 = 1.73^3 \tag{2.12}$$

Since the yielding curvature is $\phi_y = M_y/EI_{eff}$, and the ratio of the yielding moments is equal to the ratio of the effective moments of inertia ($M_{y3}/M_{y1} = 3 \bullet 1.73 = 1.73^3$) yielding curvature at the base of all columns is the same. That is compatible with the assumption that the yielding curvature depends mainly on the geometry of the cross section and the yielding deformation of the steel, which are the same in all columns.

The yielding displacement of the columns is proportional to the square of the column heights. The ratio of the yield displacements in the tallest and shortest column is:

$$\frac{\Delta_{y3}}{\Delta_{y1}} = \left(\frac{H_3}{H_1}\right)^2 = 3 \tag{2.13}$$

It is equal to the ratio of the force reduction factors. The ultimate displacement of the tallest column is $\Delta_{u3} = \Delta_{y3}$ (elastic response). The ultimate displacement of the shortest column is the same $\Delta_{u1} = 3\ \Delta_{y1} = \Delta_{y3}$.

The response of the same structure with different levels of provided strength should be interpreted in different way (see Fig. 2.5). In this case the yielding

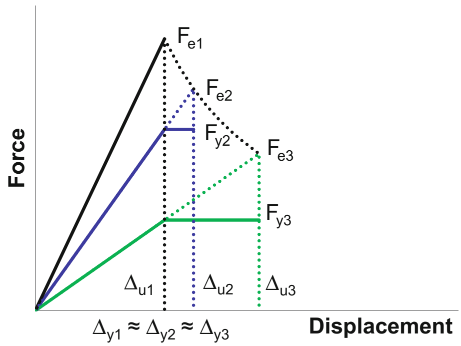

Fig. 2.5 The equal displacement rule, where correlation between the strength and the stiffness is taken into account

displacement is the same regardless of the level of provided strength, since the same structure (geometry) is addressed. This further means that the pre-yielding stiffness should be different (as illustrated in Fig. 2.5).

The ultimate displacements ($\Delta_{u1} - \Delta_{u3}$) are also different. However, this does not mean that the equal displacement rule is invalid. The seismic displacements Δ_{u2} and Δ_{u3} are still the same as those that characterize the corresponding elastic response, and are calculated taking into account the same effective pre-yielding stiffness (compare the dotted and solid lines of the same colour) and the mass. The ratio of the seismic displacements and yield displacements (e.g. ratio Δ_{u2}/Δ_{y2}) are still approximately the same as the corresponding level of force reduction (F_{e2}/F_{y2}). In other words, the equal displacement rule is valid, but it needs to be adequately interpreted, taking into account the correlation between the strength of the structure and the corresponding pre-yielding stiffness, as well as the corresponding reduced demand. It is applicable for each level of the chosen strength individually.

Basic equations of the SBD accounting for invariant yield displacement

Based on the discussion, presented in the previous section, it can be concluded that the pre-yielding stiffness is particularly important for SBD, since it defines the pre-yielding equivalent period of the structure, which further essentially influences the maximum displacement. Different procedures are proposed in the literature for estimation of this stiffness. In Eurocode 8/1 standard (CEN 2004) this stiffness is defined reducing the stiffness that corresponds to the gross cross-sections for 50%. This reduction can be adequate or not, depending on the level of the seismic force reduction. Following a similar procedure as the one presented in Fig. 2.1 but taking into account unreduced displacement spectrum (corresponding to 5% damping) it can be demonstrated that the pre-yielding stiffness of the structures with the periods in the constant velocity region of the spectrum (where it can be assumed that the reduction of seismic forces R_μ and the displacement ductility μ are equal; $\mu = R_\mu$) can be estimated as:

$$k_{eq} = \frac{4\pi^2 m \cdot S_d^2(T_D)}{(R_\mu \Delta_y)^2 T_D^2} = \frac{m \cdot S_a^2(T_D) T_D^2}{4\pi^2 (R_\mu \Delta_y)^2} \tag{2.14}$$

Based on this stiffness, and yield displacement (estimated according to Eq. 2.7), considering the relationship $\Delta_u = R_\mu \Delta_y$ ($\Delta_u = \mu \Delta_y$), the strength of the structure can be expressed as:

$$F_R = k_{eq} \Delta_y = \frac{4\boldsymbol{\pi}^2 m c_{r1}^2 S_d^2(T_D)}{T_D^2 \boldsymbol{\Delta}_u} \tag{2.15}$$

or

$$F_R = k_{eq} \Delta_y = \frac{4\boldsymbol{\pi}^2 m c_{r2}^2 S_d^2(T_D)}{T_D^2 \boldsymbol{\Delta}_u} \tag{2.16}$$

in the case without and with strain hardening, respectively. Coefficients c_{r1} and c_{r2} are the same as those presented in Sect. 2.2.1 (see *Modified DDBD taking into account equal displacement rule*).

2.2.3 Comparison of the SBD and the DDBD

Considering Eqs. 2.6, 2.10, 2.15 and 2.16 it is evident that the same strength is obtained by modified SBD and modified DDBD since the same basic assumptions are used: (a) equal displacement rule is applied in both methods, (b) strong correlation between the pre-yielding stiffness and the strength is taken into account, (c) yielding displacement does not depends on the strength.

In Fig. 2.6 the strength obtained in this way is compared to the values calculated using original DDBD. Different levels of ductility and three different values of the strain hardenings (0, 5% and 10%) were considered. It can be observed that the required strength, defined using original DDBD, is larger than that obtained by modified SBD and modified DDBD for all cases where the displacement ductility is larger than 2.5.

The SBD and DDBD were also compared using the numerical example of bridge column, presented in Fig. 2.7. Firstly, the required strength and the ultimate displacements were calculated using SBD, where the pre-yielding stiffness was estimated to be 50% of that corresponding to gross cross-section (traditional SBD). Than these quantities were estimated by modified SBD where pre-yielding stiffness was defined using Eq. 2.14. The yield displacement Δ_y was calculated as it was described in previous section. Finally the ultimate displacement and the corresponding strength were defined using DDBD and modified DDBD. All results are summarized in Table 2.1.

The largest strength of the structure was obtained using "traditional" SBD, since the pre-yielding stiffness was much larger than in other cases. Note that in this case the yielding moment and pre-yielding stiffness are not compatible. The pre-yielding

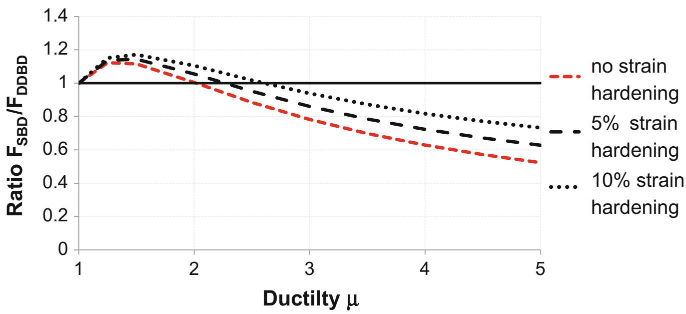

Fig. 2.6 Ratio of the strength as defined by the modified SBD (modified DDBD) and the original DDBD

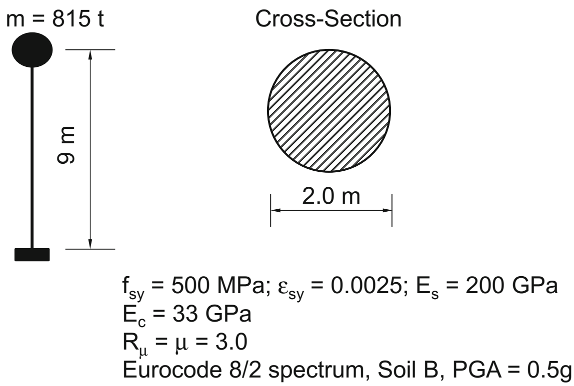

Fig. 2.7 Numerical example using the SDOF representation

Table 2.1 Summary of the results, obtained using different methods

Method	F_R (kN)	D_u (cm)	D_y (cm)	I_{eff}/I_{gross}	c_r or c_{r1}
SBD	2573	14.5	4.83	0.5	–
Modified SBD	1653	22.8	7.59	0.2	0.577
DDBD	2091	22.8	7.59	–	0.653
Modified DDBD	1653	22.8	7.59	–	0.577

stiffness of 53,300 kN/m is assumed. The equivalent pre-yielding stiffness corresponding to yield moment is 21,800 kN/m. To correlate these quantities more reliably, several iterations should be performed. After certain number of iterations the equivalent pre-yielding stiffness, defined by Eq. 2.14 was obtained. The results become the same as those of the modified SBD.

According to the previous observations modified SBD and modified DDBD provided the same results. The strength calculated using original DDBD was larger for the factor $(c_r/c_{r1})^2$:

$$\frac{F_{R,SBD}}{F_{R,DDBD}} = \frac{2091}{1635} = \left(\frac{c_r}{c_{r1}}\right)^2 = \left(\frac{\sqrt{\frac{0.07}{0.07+0.444\frac{3-1}{3\pi}}}}{\frac{1}{\sqrt{3}}}\right)^2 = 1.27 \qquad (2.17)$$

2.3 An Overview of the Pushover-Based Methods Included in the Design Codes and Guidelines

The results of the design procedures, described in the previous sections, can be evaluated using the nonlinear analysis. The most accurate non-linear method is the response-history analysis. So far it is too complex for the design practice. Therefore different simplified nonlinear methods have been developed.

The most popular are different pushover-based methods, because they explicitly take into account the nonlinearity of the seismic behaviour but at the same time considerably simplify the analysis comparing to detailed response history analysis. They can be efficiently used to examine the assumptions and the response anticipated in SBD of structures.

In the majority of modern codes and design guidelines the simplest versions of pushover based methods—single-mode methods are included. Their basic properties are described very well in (Krawinkler and Seneviratna 1998):

> The static pushover analysis has no rigorous theoretical foundation. It is based on the assumption that the response of the structure (MDOF system) can be estimated using the results of the analysis of an equivalent SDOF oscillator. This means that it is assumed that the response is governed by one invariant mode of vibration. In general this is incorrect. However, the assumption is approximately fulfilled in many (regular) structures, where the influence of the higher modes is negligible and the deflection shape is almost invariable. Thus the seismic response of these MDOF systems is quite accurately estimated based on the analysis of an equivalent SDOF model.

The first step of the majority of pushover-based methods is more or less the same. The MDOF model of the structure is pushed by lateral forces (representing the inertial forces). Their intensity is gradually increased. Shear forces and displacements are monitored and correlated forming the pushover curve.

Based on the pushover curve the properties of the equivalent SDOF system of the structure are defined and the nonlinear analysis is performed. The above procedure is common to the majority of the single-mode pushover-based methods. They differ in two ways:

- regarding the procedure that is used to estimate the properties of the equivalent SDOF model
- regarding the procedure, which is used to define the response of this equivalent SDOF system.

In general there are two different approaches, which are used to define the properties of the equivalent SDOF model. They are defined either based on the equivalent pre-yielding stiffness of the structure or based on the equivalent secant stiffness corresponding to the maximum displacement (see Fig. 2.8).

The choice of the stiffness model approach typically influences also the type of the procedure, which is used to estimate the response of the SDOF model. If the SDOF model is defined based on the pre-yielding stiffness, the maximum response of the SDOF oscillator is typically estimated based on the 5% damped acceleration spectra proposed in the codes. The target displacement of the equivalent SDOF system can be estimated using the equal displacement rule (see Fig. 2.8). Since this approximation is only suitable for the medium-and long-period structures, the displacements are corrected for short-period structures. This approach is applied in, e.g., Eurocode 8/1 (EC8/1) and implicitly in FEMA-356 (2000). In FEMA-356 (2000), the maximum seismic displacements, estimated based on the analysis of SDOF system, are additionally corrected to take into account different issues which are not included in the pushover-based analysis such as strength degradation, P-Δ effect, etc.

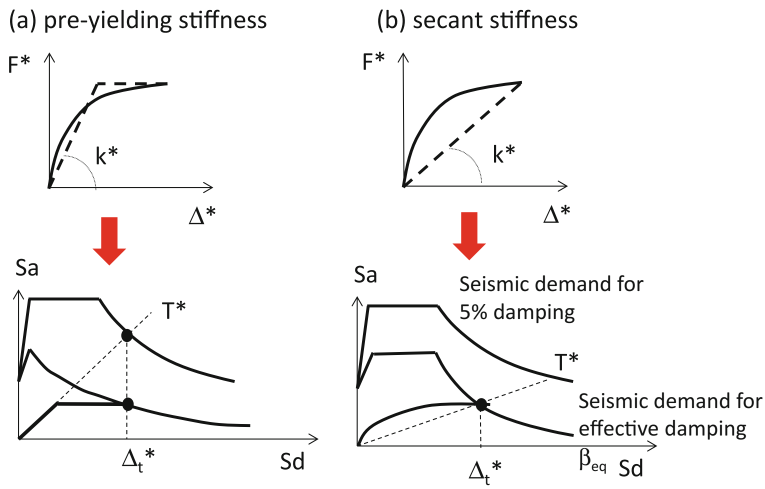

Fig. 2.8 Two different approaches, used to define properties of the equivalent SDOF system

In the second approach, where the SDOF model is defined based on the secant stiffness, the overdamped acceleration spectra are typically used (see Fig. 2.8). The capacity spectrum method approach is followed.

The application of the capacity spectrum technique means that both the structural capacity curves and the demand response spectra are plotted in the same spectral acceleration versus the spectral displacement domain and compared.

To be able to compare the capacity and demand in the same domain, the pushover curve is converted to the capacity spectrum curve using the modal shape vectors, participation factors, and modal masses obtained from a modal analysis of the structure. The capacity spectrum curve represents the relationship between accelerations S_a and displacements S_d of the equivalent SDOF oscillator. Then the standard elastic acceleration spectrum (corresponding to 5% damping) is converted to the ADRS format, where the spectral accelerations are presented as a function of the corresponding spectral displacements (see Fig. 2.8). In this way, the capacity curve and the seismic demand can be plotted on the same axes and compared.

It is assumed that the equivalent damping of the system is proportional to the area enclosed by the capacity curve. The equivalent period, T_{eq}, is assumed to be the secant period at which the seismic ground motion demands, reduced by the equivalent damping, intersect the capacity curve (FEMA-440 2005). Since the equivalent period and damping are both a function of the displacement, the solution to determine the maximum inelastic displacement (i.e., performance point) is iterative. More details about the pushover-based methods, which are included into the standards, can be found elsewhere (e.g. FEMA-440 2005; Isakovic 2014).

2.3.1 Specifics of the Analysis of Bridges

Most of the methods, which are overviewed in the previous section, have been developed primarily for the analysis of buildings. Since the response of bridges is quite different, several modifications should be introduced. They are mostly related to the construction of the pushover curve: (a) the choice of the reference location, where the displacements are monitored, (b) distribution of the lateral forces, (c) idealization of the pushover curve.

Since the detailed description of these problems and possible solutions can be found elsewhere (e.g. Kappos et al. 2012), the solutions, which are proposed at University of Ljubljana, are only summarized in this Chapter.

(a) Many standards (including Eurocode 8) propose to monitor the displacement at the centre of mass, when the pushover curve is constructed. This can be a reasonable solution as long as the location of maximum displacement coincides with the centre of mass regardless of the seismic intensity. However it should be taken into account that the location of the maximum displacement can considerably vary depending on the intensity of the seismic load. In such cases unrealistic pushover curves can be obtained if the centre of mass is chosen as a reference location. In such bridges the displacements can be considerably underestimated (see Isakovic 2014 for more details). Taking into account the extensive studies of different types of bridges (Isakovic and Fischinger 2006; Isakovic et al. 2008) performed at University of Ljubljana so far, it has been proposed to monitor the displacement at the location of the maximum displacement wherever it is.
(b) Majority of the codes suggest the use of two different distributions of the lateral load. It is recommended to take into account the envelope of the results obtained in this way. As the first option it is usually suggested to distribute the lateral load proportionally to the fundamental mode of vibration of structure.

The uniform distribution is typically suggested as the second option. In FEMA-440 (2005) it has been found that this distribution is of little value when it is used for the analysis of buildings. However, in bridges it can be useful, particularly when the higher modes have limited influence to the response (e.g. in the regions near the abutments).

In short and medium length bridges, which are supported at the abutment and which have relatively regular configuration and the response, the fundamental mode of vibration can be represented by parabolic function reasonably well. Since such distribution is easy to define, it can be used instead of the distribution proportional to the fundamental mode shape.

If the response of the bridge is governed by several modes of vibration the single-mode methods are not suitable for the analysis. The response history analysis is recommended in such cases or the multi-mode pushover based methods can be applied.

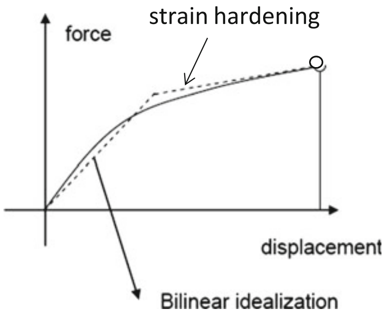

Fig. 2.9 Considerable strain hardening can occur in pushover curves of bridges pinned at the abutments

In bridges, which are not horizontally supported at the abutments, the uniform distribution of the forces seems to be the reasonable choice. However, it should be emphasized that the fundamental mode can be considerably different, depending on the stiffness and the strength of columns. Thus it is recommended to use both distributions.

In some bridges none of the distributions, described above, is the appropriate choice, particularly in structures, which are torsionally flexible (the deck of the bridge can heavily rotate in the horizontal plane, depending on the seismic intensity). In this type of bridges the response is often governed by one mode of vibration, but this mode considerably changes depending on the intensity of the seismic load. In such cases adaptive pushover-based methods should be applied.

(c) In short and medium length bridges, which are supported at the abutments, the superstructure possesses considerable stiffness after yielding of all supporting columns. As a consequence a considerable strain hardening can be observed in the pushover curve (see Fig. 2.9). In such cases the bilinear idealization of the pushover curve is recommended instead of the perfectly elasto-plastic idealization, proposed by many codes. In the case of perfectly elasto-plastic idealization, the overestimated displacements can be obtained, due to the underestimated pre-yielding stiffness of the structure. The importance of the pre-yielding stiffness is already discussed in Sect. 2.2.

In general it is recommended to perform iterative pushover analysis (one additional run is typically sufficient) since the pre-yielding stiffness depends on the achieved maximum displacement. If it is considerably different than that supposed during the idealization of the pushover curve, an additional iteration is strongly recommended.

2.3.2 Applicability of Standard Pushover-Based Methods

The main assumption of single-mode pushover-based methods is that the response of structure is governed by one invariant mode. Majority of these methods are non-adaptive. That means that they suppose that the response does not considerably change when the intensity of the seismic excitation is varied. These assumptions limit the scope of application for such methods. For example, they cannot be used for the analysis of long bridges, where the superstructure is typically quite flexible and consequently the higher modes considerably influence the seismic response.

The use of single mode methods is not recommended in bridges with very short and stiff piers, particularly if they are located close to the centre of the bridge. In such bridges higher modes are typically important, particularly for smaller intensities of the seismic load. Non-adaptive standard pushover based methods cannot be used for the analysis of torsionally sensitive structures (e.g. relatively short bridges with short and stiff central columns and with the deck, which is not supported at the abutments). More details about the applicability of these methods can be found elsewhere (e.g. Kappos et al. 2012 and Isaković 2014).

2.4 Conclusions

Two methods that can be used for the design of R/C bridges, namely the SBD and the DDBD, were analyzed in this chapter. It has been demonstrated that their results are the same when they are applied using the same set of assumptions, namely (a) the yield displacement is almost invariant for the chosen quality of the steel and the geometry of the structure, (b) the equivalent pre-yielding stiffness is strongly correlated to the strength and (c) the equal displacement rule is applied in both cases.

The strength determined by using the original DDBD, where the equal displacement rule is typically disregarded, was compared with that defined by the SBD, where the pre-yield stiffness and the strength were adequately correlated. It was found that these differences can be expressed numerically using a coefficient, which is the function of the displacement ductility μ and the type of structure. It was found that the SBD was more conservative in the case of a relatively small displacement ductility demand μ (i.e., if μ has a value of less than 2.5). In other cases, the larger required strength was obtained using the DDBD.

It has been concluded that in the SBD, the pre-yielding stiffness is directly proportional to the strength. A larger strength correlates with a larger pre-yield stiffness and vice versa. Using the typical assumption where the pre-yielding stiffness is defined to be 50% of the stiffness proportional to the gross cross-section, the equivalent stiffness as well as the required strength is often overestimated.

The equal displacement rule is also discussed. It is concluded that it should be applied for each level of the chosen strength individually, when the same structure with different strength is analyzed.

The basic assumptions and properties of the non-linear pushover-based methods, which are included into modern codes are finally reviewed. Some specifics related to their use for the analysis of bridges are presented. They are mostly related to the construction of the pushover curve as follows: (a) the choice of the reference location, where the displacements are monitored, (b) the distribution of the lateral forces and (c) the construction of the pushover curve. A short discussion on the applicability of standard pushover-based methods is also provided. It has been concluded that they can be used for the analysis of bridges where the response is governed predominantly by one mode, which is only slightly changing when the intensity of the load is varied. Typically, they cannot be used for the analysis of long bridges, where the response is highly influenced by higher modes, and for the analysis of short bridges, supported by short central columns (i.e., torsionally flexible structures), where the predominant mode of vibration changes as the intensity of the seismic excitation is varied.

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