Displacement Based Assessment for precast concrete structures: application to a three story plane frame

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SUMMARY:
Starting from the Direct Displacement Based Design (DDBD) procedure, a new assessment method is investigated in order to evaluate precast concrete structures. The proposed assessment procedure (Displacement Based Assessment – DBA) takes into account the moment-curvature and force-displacement relationship of typical precast connections, beam to column and column to foundation, to estimate the system equivalent viscous damping as a function of rotational and translational ductility of the structural elements and connections. The input data used in the assessment procedure, the displacement profile of the first inelastic mode of vibration and the system yield displacement, are obtained by means of a pushover analysis. The DBA procedure is applied to a three story precast concrete frame and validated by means of nonlinear incremental dynamic analysis.

Keywords: Precast structure; Displacement Based Assessment; Pushover.

1. INTRODUCTION

Most of the Italian precast concrete existing buildings were designed according to previous building codes, not taking into account seismic actions. It is therefore necessary to develop a quick and reliable method to assess existing buildings in order to evaluate the seismic vulnerability. In recent years seismic design has been developed considerably, especially after the introduction of innovative design approaches according to the Performance Based Design, associating limit states to seismic events with a defined probability of occurrence. The assessment procedure proposed in this paper is based on the Direct Displacement Based Design (DDBD) procedure (Priestley et al. 2007) which focuses on the overall structural and non-structural performance of a building subjected to a seismic event.

The aim of the present study is to determine how to correctly account for typical structural details of Italian precast concrete structures, like column to foundation and column to beam connections in carrying out a structural assessment. The behavior of the structural connections influences the global seismic performance of precast RC constructions and a good assessment procedure needs to take into account their resistance and deformability when subjected to horizontal forces. The Displacement Based Assessment (DBA) procedure is applied to a three story precast concrete frame structure and validated by means of nonlinear incremental dynamic analysis.

2. DISPLACEMENT BASED ASSESSMENT PROCEDURE

The Displacement Based Assessment (DBA) procedure for existing precast structures comes directly from the Direct Displacement Based Design (DDBD) procedure (Priestley et al. 2007). The first step of DBA is the definition of an appropriate inelastic deflected shape, which allows obtaining the parameters of the substitute structure, similarly to the DDBD procedure. In the proposed method, the inelastic deflected shape is derived by a pushover analysis, in order to take into account the nonlinear behavior of the structural elements and their relative connections. The curve obtained from the pushover analysis is bilinearized and the structural deflected shape corresponding to the curve yielding ($\Delta_{y,i}$) is used to evaluate the substitute structure yield displacement ($\Delta_{y,se}$) according to DDBD procedure:

$$\Delta_{y,se} = \frac{\sum m_i \cdot \left(\Delta_{y,i}\right)^2}{\sum m_i \cdot \Delta_{y,i}} \tag{2.1}$$

Where m_i is the i^{th} floor mass. The ratio between the selected target displacement, corresponding to a chosen limit state, and the yield displacement, both identified in the pushover curve, corresponds to the displacement ductility μ_Δ, which is used to calculate the substitute structure target displacement:

$$\Delta_{u,se} = \Delta_{y,se} \cdot \mu_\Delta \tag{2.2}$$

The effective mass m_{eff}, the effective stiffness k_{eff} (secant stiffness corresponding to the target displacement) and the effective period T_{eff} of the single degree of freedom substitute structure are evaluated as follows:

$$m_{eff} = \frac{\sum m_i \cdot \Delta_{u,i}}{\Delta_{u,se}}; \quad k_{eff} = \frac{V_u}{\Delta_{u,se}}; \quad T_{eff} = 2 \cdot \pi \cdot \sqrt{\frac{m_{eff}}{k_{eff}}} \tag{2.3, 2.4, 2.5}$$

The point corresponding to the obtained effective period and the target displacement lies on the damped displacement spectrum ($S_{D,in}$). In order to evaluate the return period associated to the target displacement it is necessary to obtain the elastic displacement spectrum ($S_{D,el}$). This can be done inverting Eqn. 2.6 once the equivalent viscous damping (ξ_{eq}) of the structure is determined according to DDBD procedure (an example is given in Chapter 4).

$$\frac{S_{D,in}(T_0, \mu_\Delta)}{S_{D,el}(T_{eff})} = \sqrt{\frac{10}{5+\xi_{eq}}} \tag{2.6}$$

The return period (T_R) is obtained from Eqn.2.7 by interpolation between two known T_R – PGA (Peak Ground Acceleration) couples (T_{R1}-PGA_1 and T_{R2}-PGA_2) knowing the PGA associated to the elastic displacement spectrum.

$$T_R = T_{R1} \cdot e^{\frac{\ln\left(\frac{T_{R2}}{T_{R1}}\right)}{\ln\left(\frac{PGA_2}{PGA_1}\right)} \cdot \left[\ln(PGA) - \ln(PGA_1)\right]} \tag{2.7}$$

3. PRECAST CONNECTIONS

The analysis of precast connections is fundamental for the correct evaluation of the behavior of a precast structure subjected to seismic horizontal loads: in the connection zone there is a stress and deformation concentration, influencing the global response of the building.

3.1 Column-Foundation Connections

Regarding column to foundation connections, a procedure is proposed to take into account the different yield curvature associated to different types of precast connections which will be used to estimate the displacement ductility. Priestley (2003) calculated the yield curvature of a reinforced concrete column as $2.1\varepsilon_y/B$ deriving this expression from moment-curvature analyses of square columns with flexural reinforcement evenly distributed along the perimeter, with a cross section size (B) of 160 cm and with a concrete cover to the flexural reinforcement of 5 cm. This equation does not properly describe the yield curvature of different column cross sections, especially when the effective depth is not as close to the column size as in the columns considered in the equation development. To overcome this limitation, the cross section size B has been substituted with the section effective depth d_s and the constant 2.1 with the parameter α_1:

$$\varphi_y = \alpha_1 \cdot \frac{\varepsilon_y}{d_s} \tag{3.1}$$

The parameter α_1 has been investigated by applying a least square procedure on the results of moment-curvature analyses, taking into account the influence of different variables such as the axial load ratio ν, the longitudinal reinforcement ratio ρ, the cross section dimension, the concrete cover, the concrete compressive strength, the yield steel strength and the steel overstrength ratio. Four different types of longitudinal reinforcement have been evaluated: 4, 8, 12 and 16 bars equally placed along the section sides. The equation of α_1, considering only the relevant parameters ν and ρ, may be expressed as:

$$\alpha_1 = \frac{\varphi_y \cdot d}{\varepsilon_y} = h_1 \cdot \nu + h_2 \cdot \rho + h_3 \tag{3.2}$$

The values of h_1, h_2 and h_3 obtained from a least square procedure are shown in Table 3.1 as a function of the longitudinal rebars number.

Table 3.1. Coefficient h_1 h_2 and h_3 for 4, 8, 12 and 16 rebars

Rebars number	**4**	**8**	**12**	**16**
h_1	1.94	1.11	1.22	1.97
h_2	9.18	6.50	6.30	4.30
h_3	1.39	1.69	1.69	1.18

3.2 Beam-Column Connections

Many formulations for evaluating the force-displacement relationship for different types of precast connections are available in the literature (CNR 10025, Doneux et al. 2006, Ferreira et al. 2000, Soroushian et al. 1987, Vintzeleou 1987, Tsoukantas et al. 1989). The comparison between the aforementioned expressions with experimental data (Capozzi et al. 2011, Fan et al. 2008, Ferreira et al. 2000, Kramar et al. 2010) shows a high variability of the ultimate shear strength. To the authors knowledge the moment-rotation relationship has not been analyzed yet, therefore an attempt is made in order to describe the moment-rotation behavior of typical Italian beam-column connections, which include an elastomeric pad and dowel bars (Fig. 3.1).

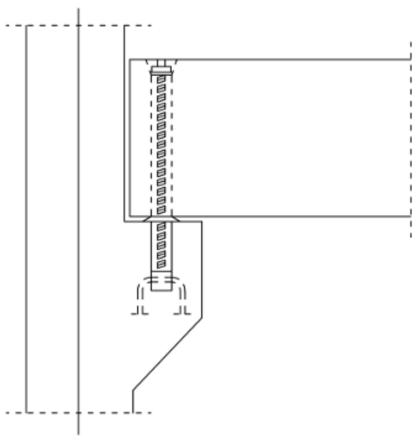


Figure 3.1. Beam-Column connection

In this study, the formulation proposed by Ferreira et al. (2000) is taken as reference for the evaluation of the force-displacement relationship. This choice is justified by the completeness of the formulation, which takes into account the post-tension of the bars and the presence of an elastomeric pad at the support. Furthermore, experimental data are available to validate the analytical results. Three different contributions are taken into account to obtain the global deformation of the connection: the deformation of the bar in the concrete embedment, the deformation of the bar in the grout embedment, the horizontal deformation of the elastomeric pad. Fig. 3.2 shows the tri-linear diagram representing the force-displacement behavior of the connection subjected to monotonic load (for the explanation of the symbols refer to Ferreira et al. 2000).

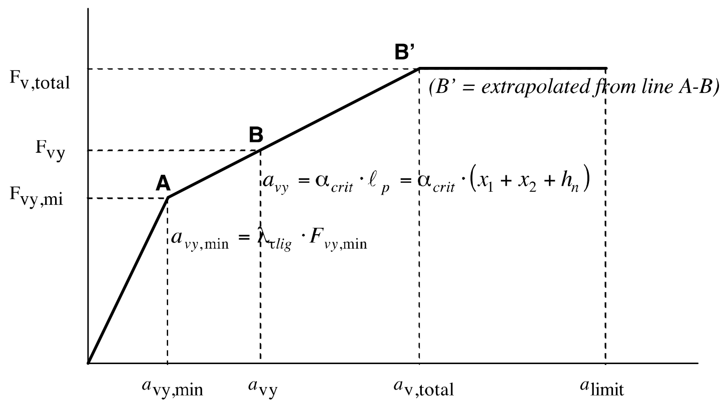


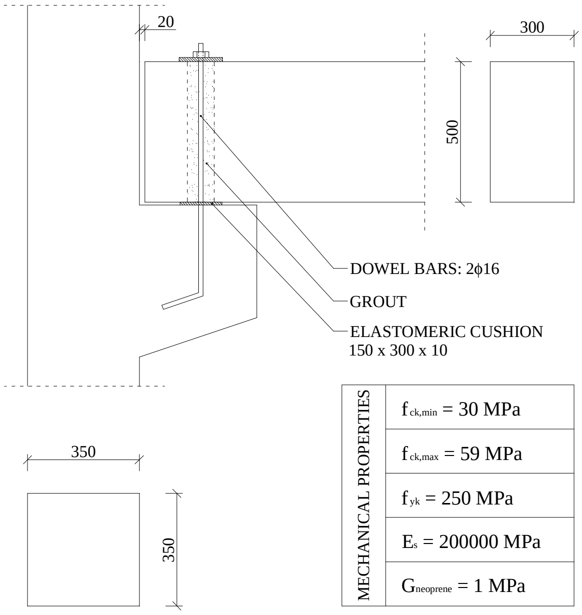
Figure 3.2. Simplified Tri-Linear Diagram used by Ferreira et al. (2000)

The comparison between the aforementioned formulation and experimental data (Ferreira et al., 2000) is shown in Fig. 3.3 for a precast beam to column connection, common in Italy, with a rubber support and two dowels, embedded in grout-filled ducts, connected to the top of the beam to avoid slippage. The comparison shows a conservative approximation of the actual behavior of the connection.

The moment-rotation diagram is calculated taking into account:

- rotation corresponding to the yielding of the dowels;
- rotation corresponding to the ultimate moment;
- rotation corresponding to an eventual contact between the top of the beam and the side of the column;
- breaking of the dowels;
- rotation corresponding to the falling of the beam from the corbel.

For a given connection, the moment-rotation relationship can be different for clockwise or counter-clockwise moments, due to the possible eccentricity of the dowel bars and the possible contact between the top of the beam and the column. An example of calculation of the moment-rotation relationship is presented in the next chapter.



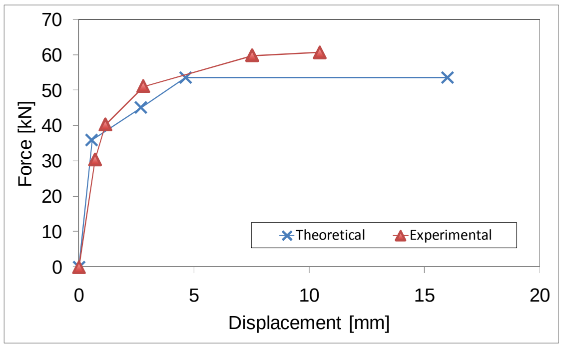


Figure 3.3. Details of the connection and force-displacement graphs

4. EXAMPLE OF CALCULATION

In this example, the simplified DBA procedure is applied to a plane frame. The frame (Fig. 4.1) represents a three-story building with three columns and six beams. The beams are jointed to the columns via dowel connections. These connections are taken into account in the model using two different non-linear hinges for each joint, describing the force-displacement and moment-rotation relationship. The columns are fixed at the ground level and soil structure interaction is not considered.

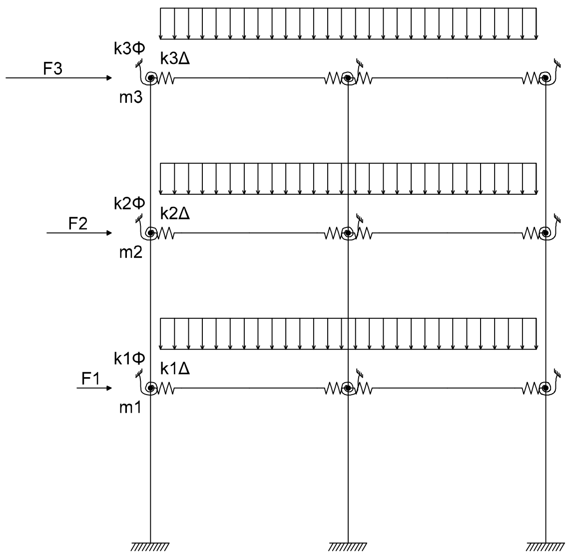


Figure 4.1. Selected frame for DBA procedure application

4.1 Beam-Column connections properties

The connection considered is the same shown in Fig. 3.4, with the addition of post-tensioning in the dowel bars up to 0.70 f_{yk}. The force-displacement relationship derived according the aforementioned formulation is presented in Fig. 4.2.

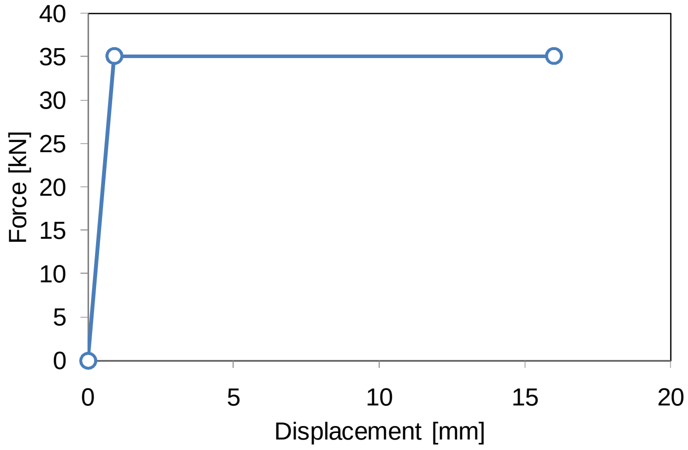


Figure 4.2. Theoretical force-displacement diagram

The moment-rotation relationship is calculated for clockwise and counter clockwise bending moments. The first step consists in the evaluation of the yield moment M_y, associated to yielding of the dowel, and of the corresponding neutral axis position 'x':

$$M_y = 6.25\ kNm;\ x = 38.37\ mm \tag{4.1, 4.2}$$

The deformation of the dowel bars associated to the applied post-tension is:

$$\varepsilon_{s,pret} = \frac{0.7 f_{yk}}{E_s} = \frac{175}{200000} = 0.0875\% \tag{4.3}$$

The corresponding section rotation is evaluated as:

$$\varphi_y = \varphi_{y,0} - \varphi_{y,pret} = \frac{\varepsilon_s \cdot l_s}{d_s - x} - \frac{\varepsilon_{s,pret} \cdot l_s}{d_s - x} = \frac{0.00125 \cdot 750}{150 - 38.37} - \frac{0.000875 \cdot 750}{150 - 38.37} = 2.52 \cdot 10^{-3} rad \tag{4.4}$$

where ε_s is the dowel yield deformation (f_{yk}/E_s). The second step consists in the calculation of the ultimate moment, the corresponding neutral axis and the dowel strain:

$$M_u = 11.08\ kNm;\ x = 10.94\ mm;\ \varepsilon_s = 2.049\ \% \tag{4.5, 4.6, 4.7}$$

the associated rotation is:

$$\varphi_{Mu} = \frac{\varepsilon_s \cdot l_s}{d_s - x} - \varphi_{y,pret} = \frac{0.02049 \cdot 750}{150 - 10.94} - 5.88 \cdot 10^{-3} = 104.63 \cdot 10^{-3} rad \tag{4.8}$$

For counter clockwise moment, as previously mentioned, the contact between the top of the beam and the side of the column can be reached. The available rotation φ_{av} before the contact is:

$$\varphi_{av} = \frac{gap}{H + h_n} = \frac{20mm}{500mm + 10mm} = 39.21 \cdot 10^{-3} rad \quad (4.9)$$

The available rotation is evaluated in order to understand if, and at which stage, the contact occurs. In this specific case the contact occurs between the yielding and the ultimate moment, therefore the moment corresponding to the contact is obtained by interpolation as:

$$\frac{M_{contact} - M_y}{\varphi_{av} - \varphi_y} = \frac{M_u - M_y}{\varphi_{Mu} - \varphi_y} \quad \Rightarrow \quad M_{contact} = 7.98 \ kNm \quad (4.10)$$

After contact is reached, there is an increase in stiffness. The moment associated to shear failure ($F_{v,tot}$) of the dowel is:

$$M_{contact,u} = F_{v,tot} \cdot 0.9 \cdot (H + e) = 35080 \cdot 0.9 \cdot (500 + 5) = 15.94 \ kNm \quad (4.11)$$

Once the dowel fails the beam can rotate until it falls from the support. The residual moment is obtained considering the friction between concrete and neoprene with the following expression (Capozzi 2008):

$$\sigma' = \frac{\sigma_{sm} \cdot n \cdot A_s}{A_0} = \frac{75 \cdot 2 \cdot 201}{45000} = 0.67 \ MPa \quad (4.12)$$

$$\mu = 0.1 + \frac{0.055}{\sigma_v} = 0.1 + \frac{0.055}{0.67} = 0.182 \quad (4.13)$$

$$M_{u,friction} = \mu \cdot \sigma' \cdot n \cdot A_s \cdot 0.9 \cdot (H + e) = 0.182 \cdot 0.67 \cdot 2 \cdot 201 \cdot 0.9 \cdot (500 + 5) = 2.50 \ kNm \quad (4.14)$$

$$\varphi_{u,fall} \cong 600 \cdot 10^{-3} \ rad \quad (4.15)$$

In Fig. 4.3 the theoretical moment-rotation relationship is presented for both clockwise (negative rotation values) and counter clockwise (positive rotation values) moments.

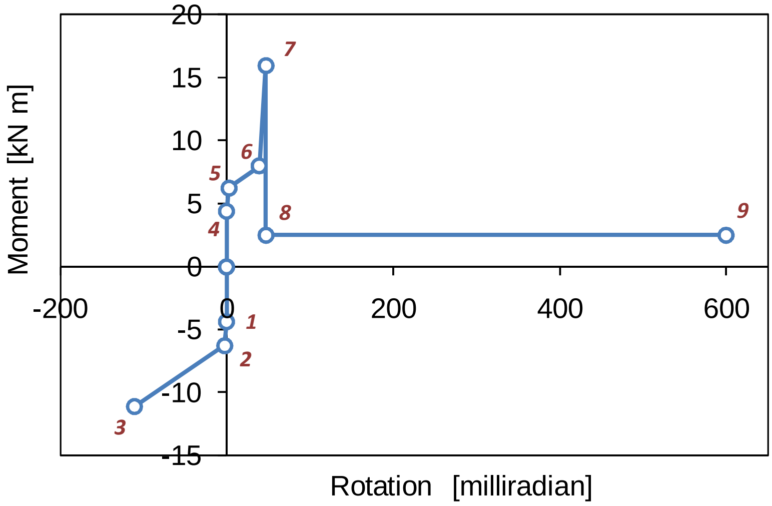


Figure 4.3. Moment-Rotation diagram

The diagram in Fig. 4.3 shows the loss of post-tension (point 1 and 4), the dowels yielding (point 2 and 5), the dowel failure in tension (point 3), the contact between the top of the beam and the column (point 6), the dowel failure in shear (point 7), the concrete-neoprene friction residual moment (point 8) and the fall of the beam from the support (point 9).

4.2 DBA Procedure

The pushover analysis of the frame is performed in order to define the inelastic deflected shape of the structure as shown in Fig. 4.5 considering three different situations: frame with pinned-pinned joints, frame with fixed-fixed joints and frame with nonlinear hinges to represent the actual nonlinear behavior of the beam-column connections.
The values of the “pinned-pinned” and “nonlinear hinges” case are quite close: the force associated to the former is about 14% less than the latter and the yielding and ultimate displacements are about the same.

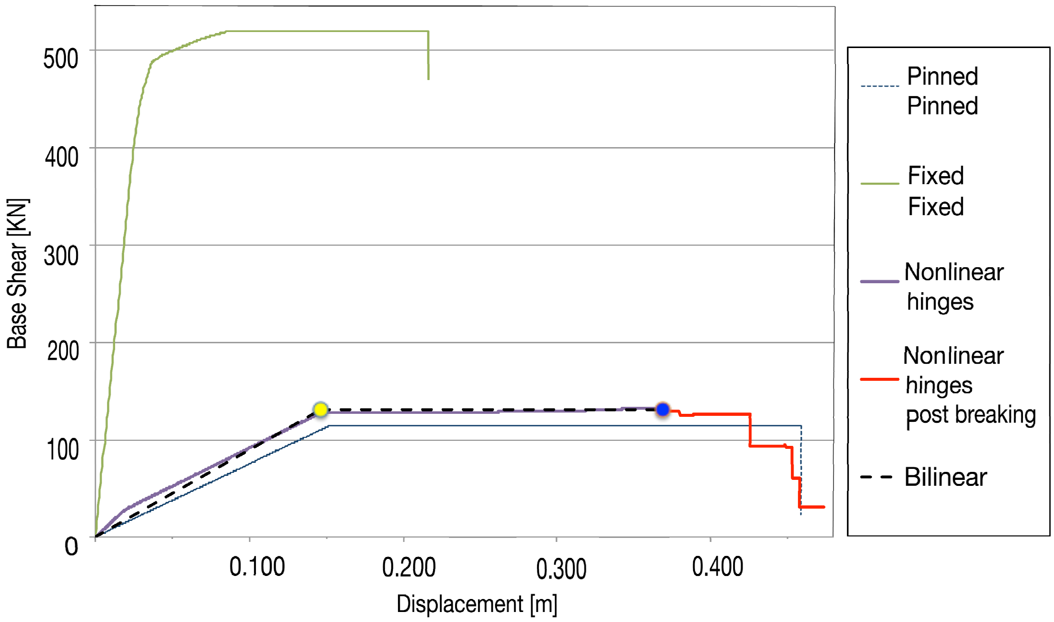


Figure 4.5. Pushover curves of the frame considered in the example

The red line in Fig. 4.5 (nonlinear hinges – Post breaking) shows the branch of the curve after the breaking of the first connection (blu circle), taken as the limit state whose return period needs to be evaluated. Therefore the main points of the pushover curve adopted in the DBA procedure are the yield (yellow circle) and the target (blu circle) displacements corresponding to Δ_y= 0.146 m V_y= 132 kN and Δ_u= 0.369 m V_u= 132 kN.
The displacement ductility (Δ_u/Δ_y) is 2.53 and the yield and target displacements of the substitute structure are calculated as:

$$\Delta_{y,se} = \frac{\sum m_i \left(\Delta_{y,i}\right)^2}{\sum m_i \Delta_{y,i}} = 0.103\ m\ ;\ \Delta_{u,se} = \Delta_{y,se}\mu_{\Delta} = 0.261\ m \qquad (4.16,\ 4.17)$$

The effective stiffness, the effective mass and the effective period are respectively:

$$k_{eff} = \frac{V_u}{\Delta_{u,se}} = 503\frac{kN}{m}\ ;\ m_{eff} = \frac{\sum m_i \Delta_{u,i}}{\Delta_{u,se}} = 113000kg\ ;\ T_{eff} = 2\pi\sqrt{\frac{m_{eff}}{k_{eff}}} = 2.97\,\text{s} \qquad (4.18,\ 4.19,\ 4.20)$$

Regarding the equivalent viscous damping evaluation, in this example the contribution of the beam-column connection has not been considered due to its negligible value compared to the one associated to plastic hinge at the column base. Therefore the equivalent viscous damping due to the columns contribution is:

$$\xi_{eq} = 0.05 + a\left(1 - \frac{1}{\mu_\Delta^b}\right)\left(1 + \frac{1}{\left(T_{eff} + c\right)^d}\right) = 12.76\% \tag{4.21}$$

With a, b, c, d according to Takeda "thin" hysteretic model (Grant et al. 2004) equal to 0.183, 0.588, 0.848 and 3.607 respectively.

The considered building is supposed located in L'Aquila (Italy) and the response spectrum (Italian building code) is associated to the following parameters: peak ground acceleration 0.261 g, soil type C, nominal building life 50 years, coefficient of use 1, topography category T1, S = 1.33, F_o = 2.364, T_B T_C T_D equal to 0.172s, 0.516s and 2.643s respectively.

The peak ground acceleration (PGA) associated to the target displacement (failure of the first connection) is:

$$PGA = \frac{\Delta_{u,es} \cdot 4\pi^2}{S \cdot F_o \cdot T_D \cdot T_C} \cdot \sqrt{\frac{\xi_{eq} + 5}{10}} = 0.327\ g \tag{4.22}$$

According to Eqn. 2.7, the return period is calculated:

$$T_R = T_{R1} \cdot e^{\frac{\log\left(\frac{T_{R2}}{T_{R1}}\right)}{\log\left(\frac{PGA_2}{PGA_1}\right)} \cdot \left[\log(PGA) - \log(PGA_1)\right]} = 917\ years \tag{4.23}$$

Finally a nonlinear incremental dynamic analysis (IDA) has been carried out with an artificial spectrum-compatible accelerogram in order to check the efficiency of the DBA procedure. The results of the IDA show that the considered limit state, failure of the beam-column connection, is associated to a PGA of 0.383 g. This value is consistent with the PGA obtained with the DBA procedure and shows that the proposed procedure leads to conservative estimations.

5. CONCLUSIONS

The paper proposes the application to a reinforced concrete precast frame of an assessment procedure (DBA) based on the Direct Displacement Based Design (DDBD). This method adopts a pushover analysis to obtain the appropriate inelastic deflected shape, which allows the definition of the parameters of the substitute structure accordingly to DDBD procedure. The DBA method is applied to a plane frame which represents a three-story precast building. The beams are jointed to the columns via dowel connections and their contribution to the global response is taken into account by means of lumped inelastic hinges describing the force-displacement and moment-rotation relationships. The equivalent viscous damping, used in the definition of the elastic displacement spectrum, is evaluated considering only the column to foundation connections due to the negligible contribution provided by the column-beam connections in the selected case study. The yield curvature equation (Priestley 2003) is reformulated and recalibrated to obtain a more suitable estimation.

The effectiveness and the accuracy of the results obtained by means of the assessment method have been preliminary validated by means of a nonlinear incremental dynamic analysis with a spectrum compatible accelerogram. The comparison shows that the proposed DBA procedure leads to conservative results.

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