A 3D continuum FE-model for predicting the nonlinear response and failure modes of RC frames in pushover analyses

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Abstract Compared to the commonly employed finite element models of RC structures in earthquake engineering, based on structural elements, refined finite element models, characterized by discretizing the concrete by 3D continuum elements together with an advanced nonlinear material model for concrete combined with 1D truss elements for the reinforcement together with an elastic–plastic material model for steel, allow valuable deeper insights into the stress distribution in RC structures and the evolution of concrete damage. As a first step towards the application of such refined finite element models in earthquake engineering, their capabilities and shortcomings are demonstrated for pushover analyses. For this purpose, pushover analyses of four RC frames were performed, for which well documented extensive test data from shaking table tests, conducted by Yavari, is available. The comparison of numerical and experimental results demonstrates the capability of refined FE-models to capture the lateral load carrying capacity as well as the location and evolution of concrete damage very well. However, the well-known shortcoming of pushover analyses of predicting a much larger lateral ductility compared to the observed one in the shaking table tests was also observed.

Keywords Reinforced concrete structure · Constitutive model · Damage-plasticity model · Finite element method · Push-over analysis · Earthquake engineering

1 Introduction

Numerical models of reinforced concrete (RC) structures within the framework of the finite element method (FEM) can be based either on structural elements (beam elements) or 3D continuum elements for the concrete in combination with 1D truss elements for the reinforcement. The former approach is computationally much cheaper and, thus, up to now it is

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preferred in earthquake engineering to simulate the structural behavior due to earthquake excitation. However, it does not deliver a detailed view of the structural response during an earthquake. The latter approach is computationally more expensive but it provides detailed information of the damage evolution in the concrete and of the stresses in the reinforcement. Hence, the application of computationally expensive continuum FE-models will be of interest for the design of large-scale tests in earthquake engineering and for partially replacing such very time-consuming and expensive tests by numerical simulations.

As a first step towards the application of refined FE-models in earthquake engineering, pushover analyses for assessing the response of RC structural components will be presented in this paper. The numerical results will be compared with experimental data for demonstrating the potentials of refined FE-models and—at the same time—recalling the limitations of push-over analyses in earthquake engineering.

For the comparison of numerical and experimental results, well documented tests are required. Up to now the majority of cyclic and dynamic tests has been carried out on RC columns. In these tests the influences of a large variety of parameters on the structural behavior were studied, like the magnitude of the axial load (Lam et al. 2003; Ousaleem et al. 2004; Matamoros et al. 2008), the influence of a variable axial force compared to a constant axial force (Lejano et al. 1992; Esmaily and Xiao 2004), the reinforcement ratio (Wibowo et al. 2014; Henkhaus et al. 2013), the amount of transverse reinforcement (Lam et al. 2003; Staaciouglu 1984), the type of cyclic loading (Lynn et al. 1996; Henkhaus et al. 2013), the effects of multi-axial loading compared to uniaxial loading (Staaciouglu 1984; Yoshimura and Tsumura 2000; Osorio et al. 2017) and of different axial and lateral load histories (Ousaleem et al. 2004; Henkhaus et al. 2013). Cyclic and/or dynamic tests on structural components with a higher degree of complexity were conducted by Pinho and Elnashai (2000), who tested a full-scale four-story three-bay RC frame, designed according to typical 1950s construction methods in southern European countries, under pseudo-dynamic loading, by Calvi et al. (2002), who conducted a quasi-static cyclic test on a three-story three-bay RC frame with non-seismic detailing, by Pinto et al. (2002), who carried out pseudo-dynamic tests on two full-scale one-bay by three-bay by four story 3D frames, bare and infilled with masonry, designed with non-seismic detailing and, more recently by Yavari (2011, 2013), who tested four two-bay two-story RC frames. The latter are selected for the present investigation because of the studied variations of design parameters and the excellent documentation of both the test setup and the experimental results.

In numerical simulations, the mechanical behavior of cohesive-frictional materials such as concrete can be described based on continuum models, continuum models with embedded discontinuities and discrete models. The first class is characterized by describing the mechanical behavior of a material point of infinitesimal volume by means of a stress–strain relation. The second class enhances the continuum description by a discontinuous displacement field for representing discrete cracks, e.g., (Feist and Hofstetter 2006; Theiner and Hofstetter 2009). The third class includes meshfree particle models, e.g., (Rabczuk and Belytschko 2007; Rabczuk et al. 2010), which have also been employed for static and dynamic loading of reinforced concrete in, e.g., (Rabczuk and Belytschko 2006; Rabczuk et al. 2008).

In this contribution, the continuum approach for modeling the mechanical behavior of concrete is pursued. For nonlinear numerical simulations of RC structures, the constitutive model for concrete is an essential ingredient. For push-over analyses a concrete model for quasi-static loading is sufficient. For this type of loading many 3D constitutive models for concrete have been proposed up to now. Most of them are based on the flow theory of plasticity, the theory of continuum damage mechanics, a combination of both theories or on the

micro-plane theory. The models take into account hardening material behavior in the pre-peak region of the stress–strain curves and softening in the post-peak region, cracking due to tensile loading and crushing due to compressive, shear or mixed mode loading. An overview over constitutive models for concrete together with a comparison of selected models is available, e.g., in (Valentini and Hofstetter 2013). In the present context, the damage-plasticity model proposed by Grassl and Jirásek (2006) is selected as constitutive model for concrete because of its good performance, numerical robustness and efficiency (Valentini and Hofstetter 2013).

In the following, at first, the employed constitutive models for concrete and reinforcing steel are reviewed briefly. Subsequently, the tests by Yavari (2011, 2013) are described, followed by the presentation of the FE-models for the push-over analyses and the comparison of the computed response with the experimental results. From the latter, conclusions on the capabilities and shortcomings of push-over analyses with refined FE-models are drawn and an outlook for future work is given.

2 Employed constitutive models for concrete and reinforcing steel

The material behavior of concrete is described by means of the damage plasticity model by Grassl and Jirásek (2006).

The model represents isotropic hardening and softening material behavior, inelastic strains and stiffness degradation due to damage by means of the combined theories of plasticity and continuum damage mechanics. The nominal stress–strain relation is given as

$$\boldsymbol{\sigma} = (1 - \omega)\mathbb{C} : (\boldsymbol{\epsilon} - \boldsymbol{\epsilon}^p), \quad (1)$$

with $\boldsymbol{\sigma}$ denoting the nominal stress tensor (force per total area), ω the scalar isotropic damage parameter ranging from 0 (undamaged material) to 1 (fully damaged material), $\mathbb{C}$ the fourth order elastic stiffness tensor, $\boldsymbol{\epsilon}$ the total strain tensor, and $\boldsymbol{\epsilon}^p$ the plastic strain tensor. The plastic part of the model is formulated in the effective stress space, with the effective stress tensor $\bar{\boldsymbol{\sigma}}$ (force per undamaged area) defined as

$$\bar{\boldsymbol{\sigma}} = \frac{1}{1 - \omega} \boldsymbol{\sigma}. \quad (2)$$

To delimit the elastic domain, a single yield surface f_p is employed. It is formulated in the Haigh-Westergaard coordinates of the effective stress tensor $\bar{\boldsymbol{\sigma}}$, i.e., the effective mean stress $\bar{\sigma}_m$, the effective deviatoric radius $\bar{\rho}$ and the Lode angle θ :

$$f_p(\bar{\sigma}_m, \bar{\rho}, \theta, q_h) = \left((1 - q_h) \left(\frac{\bar{\rho}}{\sqrt{6}f_{cu}} + \frac{\bar{\sigma}_m}{f_{cu}} \right)^2 + \sqrt{\frac{3}{2}} \frac{\bar{\rho}}{f_{cu}} \right) + m_0 q_h^2 \left(\frac{\bar{\rho}}{\sqrt{6}f_{cu}} r(\theta) + \frac{\bar{\sigma}_m}{f_{cu}} \right) - q_h^2. \quad (3)$$

Therein, f_{cu} denotes the uniaxial compressive strength, m_0 is a frictional model parameter, $r(\theta)$ is the Willam and Warnke (1975) function for describing the shape of the yield surface in deviatoric planes, and q_h is a stress-like internal hardening variable.

The evolution of the plastic strain is described by means of a non-associated plastic potential g_p as

$$\dot{\boldsymbol{\epsilon}}^p = \dot{\lambda} \frac{\partial g_p(\bar{\sigma}_m, \bar{\rho}, q_h)}{\partial \bar{\boldsymbol{\sigma}}}, \quad (4)$$

in λ which denotes the consistency parameter. The plastic potential function g_p is defined as

$$g_p(\bar{\sigma}_m, \bar{\rho}, q_h) = \left((1 - q_h) \left(\frac{\bar{\rho}}{\sqrt{6}f_{cu}} + \frac{\bar{\sigma}_m}{f_{cu}} \right)^2 + \sqrt{\frac{3}{2}} \frac{\bar{\rho}}{f_{cu}} \right)^2 + q_h^2 \left(\frac{m_0 \bar{\rho}}{\sqrt{6}f_{cu}} + \frac{m_g(\bar{\sigma}_m)}{f_{cu}} \right), \quad (5)$$

in which $m_g(\bar{\sigma}_m)$ denotes a model parameter.

The stress-like internal hardening variable q_h is related to a strain-like internal hardening variable α_p by

$$q_h = \begin{cases} \frac{f_{cy}}{f_{cu}} + \left(1 - \frac{f_{cy}}{f_{cu}}\right) \alpha_p (\alpha_p^2 - 3\alpha_p + 3) & \text{if } \alpha_p < 1, \\ 1 & \text{else} \end{cases}, \quad (6)$$

with f_{cy} denoting the uniaxial yield stress in compression. The evolution of α_p is defined by the modified hardening law according to Unteregger et al. (2015) as

$$\dot{\alpha}_p = \dot{\epsilon}^p \frac{1}{x_h(\bar{\sigma}_m)} \left(1 + 3 \frac{\bar{\rho}^2}{\bar{\rho}^2 + 10^{-8} f_{cu}^2} \cos^2 \left(\frac{3\theta}{2} \right) \right), \quad (7)$$

in which $x_h(\bar{\sigma}_m)$ is a hardening ductility measure, dependent on the effective mean stress.

Softening material behavior is represented by means of an isotropic damage parameter ω . Its evolution is defined as

$$\omega = 1 - \exp \left(-\frac{\alpha_d}{\epsilon_f} \right). \quad (8)$$

Therein, ϵ_f is the softening modulus, which is computed based on the specific mode I fracture energy G_{FI} , the characteristic size of a finite element l_{char} , the uniaxial tensile strength f_{tu} and the Young's modulus E , following the approach proposed by Valentini (2011) as

$$\epsilon_f = \frac{G_{FI}}{f_{tu} l_{char}} - \frac{f_{tu}}{2E}. \quad (9)$$

The evolution of the damage driving internal variable α_d is related to the evolution of the plastic strain as

$$\dot{\alpha}_d = \begin{cases} 0 & \text{if } \alpha_p < 1, \\ \frac{\dot{\epsilon}_v^p}{x_s(\dot{\epsilon}^p)} & \text{else} \end{cases}, \quad (10)$$

with $x_s(\dot{\epsilon}^p)$ representing a softening ductility measure dependent on the plastic strain rate, and $\dot{\epsilon}_v^p$ denoting the volumetric part of the plastic strain rate.

In addition to the material parameters for concrete, the model contains six model parameters. For the latter the values recommended in (Grassl and Jirásek 2006) are taken. The evolution of the yield surface in the principal stress space during hardening is shown in Fig. 1. The capability of the concrete model to represent the effect of confinement on the material response is shown in Fig. 2 for a triaxial compressive stress test conducted by Van Mier (1984). The confining lateral principal stresses σ_2 and σ_3 were both 10% of

Fig. 1 Evolution of the yield surface of the concrete model during hardening

Fig. 2 Comparison of observed and predicted concrete behavior in a triaxial compression test by Van Mier (1984) with confining lateral stresses of $0.10 f_{cu}$

the uniaxial compressive strength of the considered concrete, respectively. The reported material parameters for concrete by Van Mier are characterized by a Young's modulus $E=27$ GPa, a Poisson's ratio $\nu=0.2$, a uniaxial compressive strength $f_{cu}=45.3$ MPa and a uniaxial tensile strength $f_{tu}=2.8$ MPa.

Strain softening is predicted for shear and tensile loading accompanied by degradation of the elastic stiffness. A comparison of the computed model response in uniaxial tension with experimental results, reported in (Gopalaratnam and Shah 1985), is shown in Fig. 3. For the uniaxial tension test, the material parameters for concrete are given as $E=28$ GPa, $\nu=0.2$, $f_{cu}=40$ MPa, $f_{tu}=3.5$ MPa and $G_{FI}=0.055$ N/mm. In Figs. 2 and 3 the stress–strain diagrams in the softening region are computed on the basis of the characteristic length of 100 mm, which in this case is equal to the specimen length.

A conventional 1D elastic–plastic constitutive model with linear strain hardening is employed for describing the material behavior of the reinforcing steel.

3 Shaking table tests on four RC frames, conducted by Yavari

Four two-story and two-bay reinforced concrete (RC) frames on the scale of 1:2.25 were tested on the shaking table at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan from 2008 to 2009 (Yavari 2011). The test frames were selected to represent the first two stories of an existing six-story hospital building in Taiwan. Figure 4 depicts the geometry and reinforcement details for the four tested frames. According to Fig. 4, the columns and joints are referred to by the letters A, B and C and the respective story number 1 or 2, respectively. Beams and columns were designed to exhibit the strong-beam and weak-column mechanism, characteristic for older reinforced concrete buildings (Yavari 2011), designed prior to the introduction of modern seismic codes. The columns are characterized by a quadratic cross-section. Beams were constructed with a T-shaped cross section to account for the influence of slabs on the beam stiffness. In addition, a uniform vertical load with a total weight of 10 kN was applied to the beams using lead packets, which represent loading effects of one-way slabs. Additional axial column loads were applied by prestressing the columns to the shaking table by means of pressure-controlled hydraulic jacks.

Table 1 contains the designation of the tested RC frames. They differ by (1) the magnitude of the axial loads in the columns, indicated by the first letter M (moderate axial

Fig. 3 Comparison of observed and predicted concrete behavior in a uniaxial tension test by Gopalaratnam and Shah (1985)

Fig. 4 Geometry and reinforcement details for the tested frames according to Yavari et al. (2013)

Table 1 Designation of tested RC frames (Yavari 2011; Yavari et al. 2013)

Test frame	Axial load in columns	First story joints	Failure mode in columns
MCFS	Moderate	Confined	Flexure-shear
HCFS	High	Confined	Flexure-shear
MUFS	Moderate	Unconfined	Flexure-shear
MUF	Moderate	Unconfined	Flexure

loads) or H (high axial loads), (2) the design of the joints of the first story as unconfined (U) or confined (C) joints and (3) the design of the columns as non-ductile columns, referred to as flexure-shear critical columns, indicated by the letters *FS*, or as ductile columns with sufficient confinement failing in flexure, indicated by the letter *F*.

The columns of the first story were fixed to the shaking table. The restriction of the test frames to the first two stories results in an unnatural boundary condition at the top end of the columns near the joints of the second story. Hence, to prevent failure of the latter, they were confined for all test frames.

The material properties of concrete and reinforcing steel are summarized for the four test frames in Tables 2 and 3, respectively. Since for the employed concrete only the mean value of the uniaxial compressive strength f'_c , determined at the test day on cylindrical specimens, is provided in Yavari (2011) and Yavari et al. (2013), other required parameters, including the Young's modulus E_c , biaxial compressive strength f_{bu} , tensile strength f_{tu} and the specific mode I fracture energy G_{FI} , were estimated using the fib model code for concrete structures (Taerwe et al. 2013). The Poisson's ratio ν_c and the yield stress in uniaxial compression f_{cy} are assumed on the basis of similar concrete grades. They are presented in Table 2. In Table 3, the properties of the reinforcing steel, used for both the longitudinal bars and the stirrups, are summarized. E_s is the Young's modulus, f_y is the yield stress and f_u and ε_u are the ultimate stress and the respective strain.

The test frame MCFS was considered as benchmark for the evaluation and comparison of the structural behavior of the four tested frames (Yavari et al. 2013). In addition to transverse reinforcement in the joints, transverse beam stubs with a length of 200 mm, a width equal to the column width and height equal to the height of the longitudinal beam were used to provide confinement of the joints in both stories (Fig. 4). Hence, test frame MCFS is characterized by a plane of symmetry containing the beam axes and column axes. A moderate axial load of $0.2 f'_c A_g$, with A_g denoting the cross section area, was applied to the middle column of the frame and 50% of the axial load of the middle column was applied to each of the exterior columns, yielding a total axial load of approximately 550 kN. Because of the confined joints and the non-ductile

Table 2 Concrete material parameters

	f'_c (MPa)	E_c (MPa)	ν_c –	f_{cy} (MPa)	f_{bu} (MPa)	f_{tu} (MPa)	G_{FI} (N/mm)
MCFS	34.0	30,450	0.2	11.33	39.64	3.14	0.14
HCFS	34.4	30,580	0.2	11.47	40.10	3.16	0.14
MUFS	36.5	31,250	0.2	12.17	42.50	3.29	0.14
MUF	35.8	31,000	0.2	11.93	41.68	3.25	0.14

Table 3 Parameters of the reinforcing steel

	E_s (MPa)	f_y (MPa)	f_u (MPa)	ε_u –
MCFS	199,860	439	685	0.279
HCFS	199,860	439	685	0.279
MUFS	204,824	467	702	0.293
MUF	204,824	467	702	0.293

Fig. 5 Failure modes for the tested frames according to Yavari et al. (2013): **a** MCFS, **b** HCFS, **c** MUFS, **d** MUF

columns, shear and axial failure of the first-story columns due to the wide spacing of the stirrups was expected, resulting in collapse of the frame (Fig. 5a).

Test frame HCFS differs from test frame MCFS only by applying almost twice the axial loads of approximately 1000 kN in total. Collapse of the frame occurred due to failure of columns B1, C1 and A2 (Fig. 5b).

The test frame MUFS was designed to study the structural response of RC frames with unconfined joints. For this purpose all stirrups in the joints at the first story were omitted. Moreover, according to Fig. 4, at the level of the first story beam stubs were omitted at the exterior joints and transverse beam stubs were limited to only one side of the frame. Hence, the plane containing the beam axes and column axes is not a plane of symmetry for this test frame. However, out-of-plane displacements were prevented by steel frames used to brace the test frames in the out-of-plane direction by means of frictionless rollers at each beam level. Despite cracking extending from the joints into the exterior columns of the first story, the columns in the first story did not fail but shear and axial failure of columns B2 and C2, resulting in collapse of the frame, was observed (Fig. 5c).

The test frame MUF with the same geometrical properties as frame MUFS was designed to experience failure of the unconfined joints at the first story (Fig. 5d). Hence, the spacing of the stirrups in the columns was reduced from 120 mm to 40 mm to provide sufficient confinement and shear reinforcement for all columns. Pressure-controlled hydraulic jacks were used to apply moderate axial loads of the same magnitude as for frame MCFS to the center of the columns. The total axial load applied to the frames MUFS and MUF was 525 and 550 kN, respectively) Yavari (2011).

In the experimental investigation the four tested frames were subjected to the north–south component of the ground motion record from the 1999 Chi–Chi earthquake at station TCU047. The respective ground motion data is available in the PEER ground motion database (<http://peer.berkeley.edu>). The amplitude was scaled such that a peak ground acceleration of 1.1 and 1.35 g was achieved in test 1 and test 2, respectively. The observed structural behavior of the frames during the tests is documented in Yavari (2011) and Yavari et al. (2013). In particular, the location and propagation of flexural cracks and shear cracks, of yielding of the reinforcing steel, of shear and axial failure of structural components and, finally, structural collapse is reported.

4 FE-models of the test frames

The frames are discretized by 3D isoparametric serendipity continuum elements with quadratic shape functions and full numerical integration for representing the concrete. Truss elements with quadratic shape functions are used for embedding the reinforcing bars into the continuum elements. Hence, perfect bond between concrete and reinforcement is assumed. To demonstrate mesh-independence of the numerical results, two mesh sizes of 100 and 50 mm are employed for the numerical simulations. In total, the coarse FE-mesh

Fig. 6 Finite element discretization of the test frames **a** MCFS and HCFS and **b** MUFS

consists of 876 3D continuum elements with altogether 32,535 degrees of freedom. The fine FE-mesh is characterized by 7680 3D continuum elements with altogether 203,541 degrees of freedom. Figure 6 depicts the coarse FE-discretization of three of the tested frames. In this figure, for each frame the left half shows the discretized steel reinforcement, whereas the right half depicts the discretization of the concrete. Since the discretization of test frame MUF differs from the one of the frame MUFS only by the smaller spacing of the stirrups in the columns, it is not shown.

The pushover analyses of the four tested frames are performed by the finite element software package ABAQUS (2014). In the numerical simulations, first, the gravity load is applied, followed by the application of the axial loads on top of the columns. Subsequently, the pushover analyses are performed by increasing the lateral displacement of the exterior joint A2 in an incremental manner up to failure.

5 Comparison of the computed response with the experimental results

5.1 Test frame MCFS

In Fig. 7a for the frame MCFS the measured base shear is plotted in terms of the drift ratio during test 1 (blue color) and test 2 (black color), for the latter test until collapse of the frame occurred. The drift ratio is calculated from the measured lateral displacement of the second story divided by the height of the frame of 3.10 m, measured from top of the footing up to the bottom of the second-story beams (cf. Fig. 4). In the numerical simulations, the base shear is equal to the horizontal reaction force due to the prescribed lateral displacement. In addition, in Fig. 7a the response, computed by means of the pushover analysis, is depicted by the red and orange curve, referring to the coarse and fine FE-mesh, respectively. It follows from Fig. 7a that the initial stiffness and the ultimate value of the base shear are captured very well by the numerical simulations. However, due to the monotonic loading in the pushover analysis the ductility of the frame, i.e., the predicted horizontal displacement at collapse, is much larger than the observed one.

Fig. 7 Frame MCFS: **a** comparison of the measured and computed base shear and **b** computed axial forces in the first-story columns in terms of the drift ratio

Figure 7b shows the computed axial forces in the three first-story columns in terms of the drift ratio. It can be seen that with increasing drift ratio the axial force in the exterior column A is decreasing significantly, the opposite holds for the exterior column C, and the axial force in the middle column B does not change strongly. Obviously, the total axial force in the three first story columns remains constant.

The plots in Fig. 8 show the predicted damage of the frame MCFS in terms of the scalar damage variable ω for three selected values of the drift ratio. The plots in the left column show the damage of the complete frame whereas the plots in the right column show the damage after hiding the stubs at the joints, both with the center joint magnified. The first row of plots in Fig. 8 refers to the predicted slight damage at 0.30% drift ratio. At this stage, some damage is predicted at the base of the first-story columns and in the vicinity of the joints. Yielding of longitudinal reinforcing bars was observed in test 1 at 0.9% drift ratio, compared to the respective computed drift ratios of 0.75 and 0.71% for the coarse and fine FE-mesh, respectively. The second row of plots in Fig. 8 depicts the predicted damage at 1.68% drift ratio. At this stage, both the magnitude and the extent of concrete damage have increased compared to the plots, referring to 0.30% drift ratio. Whereas in the plot of the complete frame the first-story joints seem to be almost undamaged, the plot with the hidden stubs reveals considerable concrete damage within those joints. The stress distribution in the reinforcement at the same drift ratio is depicted in the left half of Fig. 9. It can be seen that small regions of the reinforcement in the vicinity of the base of the first-story columns, of the first-story joints and below the second story joints have reached or exceeded the yield stress of 439 MPa in the numerical simulation. The third row of plots in Fig. 8 illustrates the damage immediately before predicted collapse of the frame. Larger regions of severe damage are predicted at the base of the first-story columns, in the vicinity of the first-story joints and—after hiding the stubs at those joints—also at the first story joints. The respective stress distribution in the reinforcement, shown in the right half of Fig. 9, reveals that longitudinal bars of the first-story column A1 exceeded the yield stress. Finally, collapse of the frame MCFS is initiated in the numerical simulation by failure of the first-story columns at the base, as shown in Fig. 10. The latter figure provides a comparison of observed and predicted damage at the bases of the first story columns together with the computed stress in the reinforcement.

Figure 11 illustrates the evolution of the principal stresses and the respective normal stresses at two material points, which are damaged severely with increasing drift ratio. The first one is located in the right half of column A1 close to the base and inside the stirrups. The second one is located in the upper half of the first-level joint B1, again inside the reinforcement.

According to the coordinate system, shown in Figs. 8 and 9, σ_{22} in Fig. 11b is the axial stress in column A1. After application of the dead load and the additional axial loads it is a compressive stress which, however, changes to a tensile stress with increasing drift ratio, attaining quickly the tensile strength, followed by a decreasing tensile stress due to softening concrete behavior. The tensile stress in axial direction is accompanied by compressive stresses in both lateral directions. Figure 11a shows the respective principal stresses. The maximum principal stress is a tensile stress, whereas the intermediate and minimum principal stresses are compressive stresses. With increasing drift ratio the stresses in Fig. 11a, b are reduced to zero as a consequence of the progressive damage in this region of column A1.

According to the coordinate system, shown in Figs. 8 and 9, σ_{22} in Fig. 11d is the vertical stress in the first-level joint B1, σ_{33} is the horizontal stress in axial direction of the beams and σ_{11} is the out of plane stress. Immediately after application of the dead load

Fig. 8 Predicted damage of the frame MCFS before (left) and after hiding the stubs at the joints (right) for drift ratios of 0.30% (top), 1.68% (middle) and 9.84% (bottom)

Fig. 9 Predicted stress in the reinforcing steel of the frame MCFS at drift ratios of 1.68% (left) and 9.84% (right)

and the additional axial loads both σ_{11} and σ_{33} are tensile stresses, which, however, change to compressive stresses with increasing drift ratio. Similarly, the intermediate and maximum principal stress (Fig. 11c) change from tensile stresses to compressive stresses with increasing drift ratio.

Summarizing, it can be concluded that at least for certain ranges of the drift ratio tri-axial stress states are encountered, which influence the material behavior of concrete, but which would be ignored in numerical simulations with structural elements.

In Fig. 12 for the three first-story columns, A1, B1 and C1, the measured column shear is plotted in terms of the story drift ratio (horizontal displacement divided by 1.4 m) during test 1 (blue curve) and test 2 (black curve). Moreover, in Fig. 12 the response, computed by means of the pushover analyses, is depicted for both FE-meshes by the red and orange curves. Similar to the computed response of the frame MCFS (shown in Fig. 7a), it follows from Fig. 12 that for each of the three columns the initial stiffness and the maximum shear capacity are captured very well.

5.2 Test frame HCFS

Compared to the reference frame MCFS, the axial loads were doubled for the test frame HCFS. In Fig. 13 the measured base shear response of the frame HCFS during tests 1 and 2 is plotted in terms of the drift ratio. The drift ratio is calculated as described for the frame MCFS. From Fig. 13 follows that the initial stiffness and the ultimate value of the base shear are captured very well by the numerical simulations. However, again the monotonic loading resulted in a much larger ductility of the frame than the one observed in the cyclic tests.

The plots in Fig. 14 illustrate the computed damage of the frame HCFS in terms of the scalar damage variable ω immediately before predicted collapse of the frame. The plot in the left column depicts the damage of the complete frame whereas the plot in the right column shows the damage after hiding the stubs at the joints. In the left half of Fig. 14 the joints seem to be largely undamaged. The plot with the hidden stubs in the right half

Fig. 10 Observed damage at the bases of first-story columns C1, B1 and A1 of the frame MCFS (first row), computed damage immediately before predicted collapse (second row) and the stress in the reinforcement (third row); the color schemes are the same as in Figs. 8 and 9

of Fig. 14 reveals significant concrete damage within the first-story joints. The high axial load in the center column during test 2, reported in (Yavari 2011), caused axial failure of column B1 right after shear failure. Redistribution of the axial load from the center column and also local deformation at the top end of column A2 resulted in shear and axial failure of the latter column. Failure of columns B1 and A2 caused shear and axial failure of column C1 and finally collapse of the frame. A comparison of the respective observed and computed damage together with the stress in the reinforcement is provided in Fig. 15.

The first and second row of the plots in Fig. 15 reveals that large regions of concrete at the top end of columns A2 and C1 and at the base of column B1 were severely damaged. From the third row of the plots in Fig. 15, it follows that larger regions of the longitudinal

Fig. 11 Evolution of principal stresses and normal stresses at material points in the first-story column A1 and in the first-level joint B1 of the frame MCFS: **a** principal stresses and **b** normal stresses in the first-story column A1, **c** principal stresses and **d** normal stresses in the first-level joint B1

reinforcement at the top end of column A2 and at the base of column B1 exceeded the yield steel stress of 439 MPa.

5.3 Test frame MUFS

Compared to the reference frame MCFS, all stirrups in the joints at the first story were removed in the frame MUFS and the beam stubs at the exterior joints as well as transverse beam stubs at the first story were limited to one side of the frame.

The measured base shear in terms of the drift ratio for the frame MUFS during tests 1 and 2 is plotted in Fig. 16. The drift ratio is calculated from the measured lateral displacement of the second story divided by the height of the frame of 3.11 m according to (Yavari 2011), measured from top of the footing to the bottom of the second-story beams. Figure 16 illustrates that the computed ultimate value of the base shear is close to the

Fig. 12 Comparison of the measured and computed column shear in terms of the story drift for the first-story columns, A1, B1 and C1

Fig. 13 Comparison of the measured and computed base shear in terms of the drift ratio for the frame HCFS

Fig. 14 Predicted damage of the frame HCFS before (left) and after hiding the stubs at the joints (right) for drift ratio of 9.71%

measured maximum value in test 1. However, from the available plots of test data it is difficult to identify the initial stiffness of the tested frame.

The plots in Fig. 17 show the predicted damage of the frame MUFS in terms of the scalar damage variable ω immediately before predicted collapse of the frame. Since the frame MUFS is unsymmetric (cf. Fig. 4), both longitudinal views are shown in this figure. The plots in the left column depict the damage of the complete frame whereas the plots in the right column show the damage after hiding the remaining stubs at the joints. It can be seen in Fig. 17 that the first story columns exhibit damage at the bottom and at the unconfined first-level joints, the confined second-level joints remain almost undamaged, whereas below those joints the second-story columns are damaged severely. The first row of plots in Fig. 18 depicts shear and axial failure of columns B2 and C2, causing collapse of the frame MUFS, and joint A1 exhibiting the most severe damage among the first-level joints. The second row of plots in Fig. 18 shows the respective numerical results, indicating that large regions of concrete at the top end of columns B2 and C2 and of joint A1 are considerably damaged. From the third row of the plots in Fig. 18 follows that large regions of reinforcement at the top end of columns B2 and C2 exceeded the yield steel stress of 467 MPa whereas the longitudinal reinforcement at joint A1 remained in the elastic range, except for one of the longitudinal bars at the top end of column A1.

5.4 Test frame MUF

The test frame MUF differs from the test frame MUFS only by reducing the spacing of the stirrups in columns from 120 to 40 mm. Figure 19 depicts the measured base shear response in terms of the drift ratio for the frame MUF during tests 1 and 2. The drift ratio is calculated as described for the frame MUFS. It follows from Fig. 19 that the ultimate value of the base shear is somewhat overestimated by the numerical model whereas the initial stiffness of the frame is difficult to be determined from the available plots of test data.

Fig. 15 Observed damage in the first-story columns C1, B1 and second-story column A2 of the frame HCFS (first row), computed damage immediately before predicted collapse (second row) and the stress in the reinforcement (the color schemes are the same as in Figs. 8 and 9)

The plots in Fig. 20 show the predicted damage of the frame MUF in terms of the scalar damage variable ω immediately before predicted collapse in the numerical simulation. Since as the frame MUFs also the frame MUF is unsymmetric, both longitudinal views are shown in this figure. Again, the plots in the left column depict the damage of the complete frame whereas the plots in the right column show the damage after hiding the remaining stubs at the joints. The predicted damage distribution is very similar to the one for the frame MUFs (cf. Figs. 17, 20).

The frame MUF was the only frame that did not collapse during test 2, however, all first-level joints were severely damaged as shown in the first row of Fig. 21. The observed damage is represented appropriately by the numerical simulation as shown in the second row of Fig. 21. The respective stress distribution in the reinforcement, shown in the third

Fig. 16 Comparison of the measured and computed base shear in terms of the drift ratio for the frame MUFS

Fig. 17 Predicted damage of the frame MUFS at the drift ratio of 9.78%: front view (top row) and rear view (bottom row) before (left) and after hiding the stubs at the joints (right)

Fig. 18 Observed damage at the top end of the first-story columns C2, B2 and first-level joint A1 (first row), computed damage in columns C2 and B2 and first-level joint A1 immediately before predicted collapse of the frame MUFS (second row) and the stress in the reinforcement (third row); the color schemes are the same as in Figs. 8 and 9

row of Fig. 21, illustrates that the stresses in the reinforcing bars at first-level joints and adjacent columns remain largely in the elastic range.

6 Summary and conclusions

As a first step towards the application of refined finite element models of RC structural components in earthquake engineering, the capabilities and shortcomings of such models were demonstrated for pushover analyses. The refined FE models are characterized

Fig. 19 Comparison of the measured and computed base shear in terms of the drift ratio for the frame MUF

Fig. 20 Predicted damage of the frame MUF at the drift ratio of 9.82%: front view (top row) and rear view (bottom row) before (left half) and after hiding the stubs at the joints (right half)

Fig. 21 Observed damage at the first-level joints C1, B1 and A1 (first row), computed damage in joints C1, B1 and A1 immediately before predicted collapse of the frame MUF (second row) and the stress in the reinforcement (third row); the color schemes are the same as in Figs. 8 and 9

by discretizing the concrete by 3D continuum elements and the reinforcement by 1D truss elements, embedded into the concrete. The nonlinear material behavior of concrete, characterized by strain hardening in the pre-peak region of the stress–strain curves and strain softening in the post-peak region, accompanied by irreversible deformation and eventually resulting in material failure, is described by an advanced 3D constitutive model (Grassl and Jirásek 2006; Valentini and Hofstetter 2013) and a conventional 1D elastic–plastic constitutive model with linear strain hardening is employed for the reinforcing steel.

Such refined FE-models were generated for four RC frames, for which well documented extensive test data from shaking table tests, conducted by Yavari (2011, 2013), is available. The tests were carried out for assessing the behavior of older RC buildings, designed to

exhibit the strong-beam and weak-column mechanism, during earthquake excitation. In the tests the magnitude of the axial loads acting in the columns, the arrangement of stirrups in the joints and the spacing of the stirrups in the columns was varied.

The numerical results were compared to the experimental results (Yavari 2011) in diagrams, depicting the base shear in terms of the drift ratio and by plots showing the evolution of concrete damage and of the stress in the reinforcement, both in terms of the drift ratio. The comparison reveals the capability of refined FE-models to capture the lateral load carrying capacity as well as the location and evolution of concrete damage very well. In addition, valuable insights into the stress and damage distribution of interior parts of the RC frames are provided. In particular, some joints of the tested RC frames, confined by stubs, seemed to be largely undamaged at the surface, whereas, after hiding the stubs in the numerical model, considerable damage in the interior of the joints was detected. However, the well-known shortcoming of pushover analyses of predicting a much larger lateral ductility compared to the observed one in the shaking table tests was also observed. Thus, the concrete model will be extended for describing cyclic loading in the next step.

Unfortunately, in the extensive experimental program on the four RC frames the only provided material parameter for concrete is the uniaxial compressive strength. Hence, the other material parameters required for the concrete model had to be estimated. Since particularly the nonlinear material behavior of concrete is influenced by further parameters, it is recommended to determine those parameters in future large-scale shaking table tests on RC structural components. Furthermore, the number of specimens employed for determining the material parameters together with the respective mean values and standard deviations should be reported. They are required for studying the influence of uncertain input parameters on the structural response of the frames. The additional effort is small compared to the large effort for conducting and documenting large-scale shaking table tests.

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