Effectiveness of ACI-318 and TS-500 Codes on Nonlinear Seismic Analysis of RC structures: Force-Based Approach

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Effectiveness of ACI-318 and TS-500 Codes on Nonlinear Seismic Analysis of RC structures: Force-Based Approach

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The Effectiveness of ACI-318 and TS-500 Codes on Nonlinear Seismic Analysis of RC structures: Force-Based Approach

ABSTRACT

Different equations for the material properties of concrete are suggested in several national codes. The effectiveness of these equations on the nonlinear seismic analysis of RC structures for the material properties has been an important research subject. In this study, the effectiveness of ACI-318 and TS 500 design codes is investigated in the numerical modeling of RC structures. For this purpose, the seismic responses of two RC structures experimented in the laboratory are used. The first structure is a two-dimensional three-bay and four-story RC bare frame structure that was tested in European Laboratory for Structural Assessment. The second structure is an RC tall bridge pier that was tested at Pacific Earthquake Engineering Research Center. The numerical solutions are obtained by using the Force-based Fiber Element Approach. The experimental and numerical analysis results are compared in terms of top displacements and damage zones. The concrete material properties which provide the best approximation to the experimental results are investigated for both design codes. It is concluded that the material properties of RC structures determined according to the ACI-318 code can be used for the Force-based Fiber Element Approach with regard to displacement response and damage regions.

KEYWORDS Reinforced concrete, Nonlinear seismic analysis, Force-based fiber element approach, Top displacement, and damage zone
1 INTRODUCTION

The seismic behavior of RC structures under earthquake loading is commonly evaluated by numerical analysis methods. In the analysis, the determination of the structural properties is the first step of the modeling. The material properties used in numerical analysis for RC structures are generally taken from the experimental test results such as the compressive test of the cylinder or cube specimens for concrete, uniaxial tensile test for steel reinforcement, etc. In particular, the uniaxial compressive strength of concrete is the most common material property used for the calculation of other structural parameters such as the elastic modulus and tensile strength of concrete. In several design codes for RC structures, different equations are used for the calculation of these parameters based on the compressive strength of concrete. (ACI-318 2008; Eurocode 2 2005; Iranian National Building Code 2014; TS-500 2000). These formulations were developed under quasi-static or static loading. When determining the seismic performance of RC structures, the efficiency of the material properties defined according to design codes has been an important research subject in the nonlinear seismic analysis of RC structures (Ariglu et al. 2006; Oluokun 1991; Sucharda et al. 2020; Peng et al. 2019).

The numerical analysis methods to obtain the nonlinear seismic response of RC structures are proposed in the literature by using various models. These models are categorized as follows; Lumped plasticity models, Distributed plasticity models, and Microscopic finite element models (Taucer et al. 1991). In the Lumped plasticity models, deformations and cracks that arise due to an earthquake loading are assumed to be only in the particular region (part) of an RC structural element. The nonlinear plastic behavior is considered for these particular regions where the moment has relatively high values. These regions are generally observed at the portions near the end of the structural elements. The other regions of the element are also assumed as linearly elastic. The model is used to predict the approximate response of structures due to limited regions determined in the calculation of the whole structure responses (Taucer et al. 1991; Li et al. 2011).

The Distributed plasticity model is based on the assumption that the deformations have occurred throughout an RC structural element. Although the model requires more effort for numerical calculations, it is preferred in most design codes due to its accuracy in the solutions depending on the development of computer technology (Lu et al. 2013; Vafaei et al. 2020). Therefore, an investigation of the model's effectiveness is required for different design codes. In the distributed plasticity models, concrete and steel reinforcement portions in the cross-section of each structural element are modeled with fiber elements. The Fiber Element Approach (FEA) is based on distributed plastic hinge method. This approach depends on the assumption that the plane sections in an element are assumed to remain plane during the deformation history (Taucer et al. 1991). The behavior of fiber elements
is used to determine the constitutive relation of the integration points (cross-section) of structural elements. FEA is divided into two methods as Displacement-based and Force-based formulations (Martinelli et al. 2013; Li et al. 2016; Vásquez et al. 2016; Feng and Xu 2018; Li et al. 2018).

In microscopic finite element models, structural members and joints are discretized into a large number of finite elements. Numerical solutions of the RC structures employing this modeling method are analyzed in detail. Moreover, physical nonlinearities such as bond deterioration between concrete and steel, creep, and thermal phenomena can be defined with this modeling type. Due to this method being computationally expensive for nonlinear analysis, it is used only in research for critical parts of large frames and structures (Taucer et al. 1991).

In this study, the effectiveness of two design codes is investigated in numerical modeling of nonlinear seismic response of RC structures. The elastic modulus and the uniaxial tensile strength are obtained by using equations proposed that depend on the uniaxial compressive strength of concrete in the American Design Code (ACI-318) and the Turkish Design Code (TS-500). In the numerical solutions, the Force-based Fiber Element Approach is applied by using SeismoStruct software (Seismosoft 2016). The experimental studies of a two-dimension three-bay four-story bare frame building (BFB) that was tested in the European Laboratory for Structural Assessment (ELSA) (Pinto et al. 2002) and a 7.32 m tall bridge pier structure (BPS) that was tested in the Pacific Earthquake Engineering Research Center (PEER) (Schoettler et al. 2015) are used for the numerical model. The experimental and numerical analysis results are compared in terms of top displacements and damage zones. The material properties of the cases which provide the best approximation to the experimental results are investigated for each design code.

2 NUMERICAL MODELLING OF THE RC STRUCTURAL ELEMENTS

Due to the complexity of interaction between various components of the RC structures, several models for the nonlinear response analysis of these structures have been proposed to date. In the analyses, Lumped plasticity models, Distributed plasticity models, and Microscopic finite element models (Taucer et al. 1991) are used.

In this research, distributed plasticity model is employed for the nonlinear analysis of RC structures. In the model, interconnected elements that are called “Fiber” are used to represent the hysteretic behavior of a structural element. The method is named as “Fiber Element Approach (FEA)”, which has two main principle formulations used displacement-based stiffness approach and force-based flexibility approach (Karaton and Awla 2018). Uniaxial stress-strain relationships of the response of each concrete and steel fiber are separately used for the nonlinear behavior of a cross-section of RC elements, after calculations, obtained responses are superposed.
Sinha et al. (1964), Karsan and Jirsa (1969), Yankelevsky and Reinhardt (1987), and Mander et al. (1988) are researchers that exert effort to obtain suitable stress-strain relations of concrete under cyclic loading. Mander et al. (1988) proposed a stress-strain model for concrete subjected to uniaxial reversed compressive and tension loading for members confined by transverse reinforcement (Fig. 1(a)). The authors used a modified equation of Popovics (1970) for monotonic compressive loading. The unloading curves were derived by the parameter adjustment based on selected experimental unloading curves for confined and unconfined concrete. For the reloading curve, a linear stress-strain relation is constructed between the point of zero stress and the unloading strain, and a parabolic transition curve is used between the unloading strain and the return to the monotonic stress-strain. Martinez-Rueda and Elnashai (1997) modified the model for taking into account the effect of degradation in stiffness and strength due to cyclic loading.

![Fig. 1](image)

(a) Mander-Priestley-Park (1988) Concrete model and (b) Menegotto-Pinto (1973) steel model

The strain-stress relations of steel reinforcement under cyclic or dynamic loading have an important effect on RC section response in the Fiber Element Approach. Therefore, a lot of approaches have been proposed in the literature (Park et al. 1972; Aktan et al. 1973; Ma et al. 1976; Filippou et al. 1983; Chang and Mander 1993). Menegotto and Pinto (1973) proposed a model for steel material under cyclic loading. The steel model was developed by Yassin (1994) based on the model proposed by Menegotto and Pinto (1973) and coupled with the isotropic hardening rules proposed by Filippou et al. (1983). The uniaxial stress-strain behavior of the model is given in Fig. 1(b).

The force-based formulation of the Fiber Element Approach is depending on a three-dimensional law for the steel and concrete fiber elements in the cross-section. In numerical solutions for the structural elements, the deformation interpolation functions depending on internal forces change during the iterative solution. Therefore, the method utilized in this study is called as Force-based Fiber Element Approach. The force field is expressed by
using a section force vector where two bending moments \(M_z(x)\) and \(M_y(x)\) are linear and axial force \(N(x)\) is constant. The section stiffness matrix is obtained by using the tangent modulus of elasticity, areas, and coordinates of each fiber at an integration point. The tangent elasticity modulus of each reinforcement or concrete fiber can be obtained using the uniaxial stress-strain relation of each material. In the section, the linear superposition principle is used to incorporate the different fiber material properties. The section tangent stiffness matrix can be expressed as in Eq. (1). In this equation, where, superscript \(j\) is the iteration number at the element level, \(i\) denotes the iteration number of the Newton-Raphson iteration loop at the structural level, \(E_{ifib}^j\) is the tangent elasticity modulus of the fiber, \(A_{ifib}\) is the area of the fiber, \(y_{ifib}\) and, \(z_{ifib}\) are the distance between section centroid and fiber centroid (Taucer et al., 1991).

\[
k^j(x) = \begin{bmatrix}
\sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot y_{ifib}^2 & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot y_{ifib} \cdot z_{ifib} & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot y_{ifib} \cdot y_{ifib} \\
\sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot y_{ifib} \cdot z_{ifib} & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot z_{ifib}^2 & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot z_{ifib} \cdot z_{ifib} \\
- \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot y_{ifib} & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib} \cdot z_{ifib} & \sum_{ifib=1}^{n(ifib)} E_{ifib}^j \cdot A_{ifib}
\end{bmatrix}
\]

(1)

3 COMPARISON OF EXPERIMENTAL AND NUMERICAL ANALYSIS RESULTS

In this study, experimental test results of a two-dimension three-bay four-story bare frame building (BFB) and bridge pier structure (BPS) were used for the comparison of numerical analysis results. The Force-based Fiber Element Approach was utilized for the numerical analysis. According to both American Standard Code (ACI-318) and the Turkish Standard Code (TS 500), the elasticity modulus \(E_c\) and tensile strength \(f_{ct}\) of concrete were determined based on the uniaxial compressive strength \(f_{cc}\) of concrete. \(E_c\) and \(f_{ct}\) parameters were calculated by using Eqs. (2)-(3) for ACI-318 and TS-500, respectively. The different values for \(f_{cc}\) were selected to investigate the effectiveness of the equations proposed by the ACI-318 and TS-500 codes. The numerical analyses were achieved by using obtained material properties and comparisons with the experimental results were performed.

\[
f_{ct} = 0.5563 \sqrt{f_{cc}} \quad \text{(MPa)}
\]

(2.a)

\[
E_c = 4700 \sqrt{f_{cc}} \quad \text{(MPa)}
\]

(2.b)

\[
f_{ct} = 0.35 \sqrt{f_{cc}} \quad \text{(MPa)}
\]

(3.a)

\[
E_c = 3250 \sqrt{f_{cc}} + 14000 \quad \text{(MPa)}
\]

(3.b)
In the numerical solutions, the Force-based Fiber Element Approach is applied by using SeismoStruct software. The Hilber-Hughes-Taylor-α (HHT-α) method is used as a dynamic integration method. Mander-Priestley-Park’s (1988) Concrete model and Menegotto-Pinto’s (1973) steel model for concrete and steel material, respectively. In the first phase of the numerical solutions, various material classes are predicted for each design code. Damage regions are defined and categorized according to a predefined damage scale for the cracking or crushing regions of the structure. In the next phase, some peak amplitudes of displacement response of the structure obtained from seismic or dynamic loading are determined and compared with numerical results obtained by using the predicted material class. A similar procedure is applied to the magnitude of damages in the structure. Differences in peak displacement values, and damage magnitudes, are calculated by Eqs. (4) and (5) as,

\[
\Delta_{\text{dis}} = \frac{\sum_{i=1}^{n} \delta_{\text{exp},i} - \delta_{\text{num},i}}{n}
\]

\[
\Delta_{\text{dim}} = \frac{\sum_{j=1}^{m} d_{\text{exp},j} - d_{\text{num},j}}{m}
\]

where, \( n \) and \( m \) are sample point number of displacement and damage magnitude number of the cracking or crushing regions into the structural elements, respectively. \( \delta_{\text{exp},i} \) and \( \delta_{\text{num},i} \) are values of \( i^{th} \) the peak point on the displacement time history graph obtained experimentally and numerically, respectively. \( d_{\text{exp},j} \) and \( d_{\text{num},j} \) are \( j^{th} \) damage magnitude obtained experimentally and numerically, respectively.

3.1. A Full-Scale Four-Story Bare Frame Building

A full-scale four-story bare frame building (BFB) was tested in the ELSA laboratory (Joint Research Centre, Ispra) in the framework of the ICONS project (Pinto et al. 2002). The building is an RC 4-story with 3-bays one of 2.5 m span and two of 5 m span (Fig. 2). The thickness of the slab is 0.15 m and the inner-story height is 2.7 m. Elevation and plan of the building, characteristics of RC materials, geometric properties of the structural elements, and all related data can be found in the test report (Pinto et al. 2002). In the numerical analysis, columns and beams are modeled through a 3D frame element by using the Force-based Fiber Element Approach. The number of integration points (sections) in the element is selected as 4 and the number of fibers in the section is selected as 200. In the comparison of experimental and numerical analysis results, the displacement response of the BFB obtained under 975-year return period acceleration loading is used (Fig. 3).

Two pools of material property are constituted by using the mechanical properties of five different concrete material classes according to ACI-318 and TS-500 codes. The obtained mechanical properties can be seen in
Tables 1-2. For the calibration of differences between the experimental and numerical analysis results, sixteen maximum peak displacement points in the time-history graphs are selected for each case (Fig. 3).

![Fig. 2 4-story RC BFB with 3-bays (Pinto et al. 2002)](image)

![Fig. 3 Selected maximum peak displacement points for comparison in the time history graphs of the BFB](image)

<table>
<thead>
<tr>
<th>Table 1 Concrete material properties according to ACI-318 Code for the BFB</th>
</tr>
</thead>
<tbody>
<tr>
<td>compressive str. of concrete</td>
</tr>
<tr>
<td>(MPa)</td>
</tr>
<tr>
<td>case-1</td>
</tr>
<tr>
<td>case-2</td>
</tr>
<tr>
<td>case-3</td>
</tr>
<tr>
<td>case-4</td>
</tr>
<tr>
<td>case-5</td>
</tr>
</tbody>
</table>

In the first loading phase, damage regions obtained from the numerical analysis could not be compared with the experimental result because slight damage regions were seen in laboratory tests under the 475yrp loading type. Significant damages were observed under 975yrp acceleration loading. In the experimental study, the test
was terminated after around 7.5 seconds to prevent the total collapse of the structure because the failure of the 3rd story was imminent at the time (Pinto et al. 2002). For this reason, the comparison of experimental and numerical model results in terms of damage regions is performed for the first 7.5 seconds under the 975yrp loading.

### Table 2 Concrete material properties according to TS 500 Code for the BFB

<table>
<thead>
<tr>
<th>Case</th>
<th>Compressive Str. of Concrete (MPa)</th>
<th>Tensile Strength of Concrete (MPa)</th>
<th>Modulus of Elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.0</td>
<td>1.106</td>
<td>24277.402</td>
</tr>
<tr>
<td>2</td>
<td>12.0</td>
<td>1.212</td>
<td>25258.330</td>
</tr>
<tr>
<td>3</td>
<td>14.0</td>
<td>1.309</td>
<td>26160.386</td>
</tr>
<tr>
<td>4</td>
<td>16.0</td>
<td>1.400</td>
<td>27000.000</td>
</tr>
<tr>
<td>5</td>
<td>18.0</td>
<td>1.485</td>
<td>27788.582</td>
</tr>
</tbody>
</table>

In this study, five damage levels by using concrete strain are introduced according to damages obtained in cross-sections of any element. These damage levels are Light, Minor, Moderate, Heavy, and Severe damages as seen in Fig. 4.c. A ratio that depends on the outer compressive strain of the cross-section and ultimate compressive strain is used. In this study, Light, Minor, Moderate, Heavy and Severe damages are determined for 0-20%, 20-40%, 40-60%, 60-80%, and, 80-100% ratios, respectively. The ultimate compressive strain is calculated by Eq. (6) (Paulay and Priestly 1992).

\[
\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{mu}}{f_{cc}} \tag{6}
\]

where, \( \varepsilon_{cu} \) and \( \varepsilon_{mu} \) are the ultimate compressive strain and maximum tensile strain of concrete and steel, respectively. \( f_{yh} \) is the yield strength of the stirrups, \( \rho_s \) is also the volumetric ratio of confining steel, and \( f_{cc} \) is the strength of confined concrete.

![Fig. 4](image_url) Cracking zones and relative rotation values of the BFB (Pinto et al. 2002)

In the experimental test, cracking zones and relative rotations obtained from the inclinometer devices were given in Fig. 4.a and 4.b, respectively. Damage regions of the lower and upper sides of Column-2 at the third story
and lower side of Column-2 at the first story were defined as heavy, severe, and severe damage in the report of the ELSA test, respectively. Additionally, it was determined that relative rotations of the lower and upper sides of Column-1 and Column-3 at the third story were in a heavy damage region. The damages that occurred on the upper side of Column-4 at the third story were assumed as moderate. Other frame elements were in the no damage, light damage, and minor damage regions. Categorized damage regions of the frame were also given in Fig. 4.c.

### 3.1.1. Investigation of the BFB according to ACI-318 code

Top story displacement time history graphs of numerical and experimental results for Case-1, Case-2, Case-3, Case-4, and Case-5 according to ACI-318 were given in Fig. 5. Mean differences between displacements obtained from experimental and the Case-1, Case-2, Case-3, Case-4, Case-5 numerical solutions were calculated as 97.1%, 58.5%, 20.3%, 15.6% and 19.1% by using Eq. (4), respectively. The minimum difference was obtained for the Case-4 solution.

![Comparison of top story displacement time history graphs](image)

**Fig. 5** Comparison of top story displacement time history graphs of numerical and experimental results for the BFB according to ACI-318 code

Damage regions obtained according to ACI-318 under earthquake loading with 975 year return period were given in Fig. 6. Differences between categorized damage ratios of experimental and the Case-1, Case-2, Case-3,
Case-4, and Case-5 numerical solutions were calculated as 19.4%, 16.3%, 16.9%, 13.1%, and 14.4% by using Eq. (5), respectively. The minimum difference was obtained in the Case-4 solution.

![Damage regions of the BFB according to ACI-318 code for all cases](image)

The comparisons of the numerical analysis results obtained by using ACI-318 code provisions with the experimental test results were given in Table 3 for the material properties. The differences in compressive strength and elasticity modulus of concrete are determined as 1.84% and 0.92%, respectively. However, the difference in the tensile strength of concrete was founded at 17.1%. It was seen that the result of Case 4 has the best agreement with the ELSA test results.

<table>
<thead>
<tr>
<th>Table 3 Comparison of material properties obtained according to ACI-318 for the BFB</th>
</tr>
</thead>
<tbody>
<tr>
<td>compressive strength (MPa)</td>
</tr>
<tr>
<td>-----------------------------</td>
</tr>
<tr>
<td>Numerical analysis</td>
</tr>
<tr>
<td>Laboratory test results</td>
</tr>
<tr>
<td>Difference</td>
</tr>
</tbody>
</table>

3.1.2. Investigation of the BFB according to TS 500 code

Top story displacement time history graphs of numerical and experimental results for Case-1, Case-2, Case-3, Case-4, and Case-5 according to TS 500 were given in Fig. 7. Mean differences between displacements obtained...
from experimental and the Case-1, Case-2, Case-3, Case-4, Case-5 numerical solutions were calculated as 97.1%, 63.5%, 45.5%, 28.6% and 30.5% by using Eq. (4), respectively. The minimum difference was obtained for the Case-4 solution.

![Fig. 7 Comparison of top story displacement time history graphs of numerical and experimental results for the BFB according to TS 500 code](image)

**Fig. 7** Comparison of top story displacement time history graphs of numerical and experimental results for the BFB according to TS 500 code

Damage regions obtained according to TS 500 code under earthquake loading with 975 year return period were seen in Fig. 8. Differences between categorized damage ratios of experimental and the Case-1, Case-2, Case-3, Case-4, Case-5 numerical solutions were calculated as 18.8%, 16.3%, 15.6%, 11.88% and 13.12% by using Eq. (5), respectively. The minimum difference was obtained in the Case-4 solution.

The comparisons of the numerical analysis results obtained using TS 500 code provisions with the experimental test results were given in Table 4 for the material properties used in the solutions. The difference was defined as 1.84% for compressive strength. However, differences in tensile strength and elasticity modulus were found at 26.3% and 42.3%, respectively. It was seen that the result of Case 4 has the best agreement with the ELSA test results.
Fig. 8 Damage regions of the BFB according to TS 500 for all cases.

Table 4 Comparison of material properties obtained according to TS 500 for the BFB

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Compressive Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Elasticity Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Numerical analysis</td>
<td>16.0</td>
<td>1.4</td>
<td>27000</td>
</tr>
<tr>
<td>Laboratory test results</td>
<td>16.3</td>
<td>1.90</td>
<td>18975</td>
</tr>
<tr>
<td>Difference</td>
<td>1.84%</td>
<td>26.3%</td>
<td>42.3%</td>
</tr>
</tbody>
</table>

3.2. A Full-Scale Bridge Pier Structure

A full-scale 7.32m tall bridge pier was selected as a second study for the verification of the code provisions. The diameter of the circular pier was 1.22 m. The Bridge Pier Structure (BPS) was tested in the Pacific Earthquake Engineering Research Center (PEER) under ten consequent artificial accelerations respectively by using shaking table dynamic testing (Fig. 9). The response of the BPS was recorded in terms of top displacement, base shear, etc. Damaged regions with the level of damage were also recorded after each acceleration loading. The results of the 3rd earthquake test were used because in that test significant damage occurred. Detailed information about material properties, geometric of the BPS, and all related data were given by Schoettler et al. (2015).

In the numerical analysis, the BPS was modeled with a 3D frame element by using the Force-based Fiber Element Approach. A material pool is achieved by using the mechanical properties of four different concrete material classes obtained according to ACI-318 and TS-500 codes. These material properties were given in Tables 5-6.
**Table 5** Concrete material properties pier according to ACI-318 code for the BPS

<table>
<thead>
<tr>
<th></th>
<th>compressive str. of concrete (MPa)</th>
<th>tensile strength of concrete (MPa)</th>
<th>modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>case-1</td>
<td>26</td>
<td>2.84</td>
<td>23965</td>
</tr>
<tr>
<td>case-2</td>
<td>28</td>
<td>2.94</td>
<td>24870</td>
</tr>
<tr>
<td>case-3</td>
<td>30</td>
<td>3.05</td>
<td>25743</td>
</tr>
<tr>
<td>case-4</td>
<td>32</td>
<td>3.15</td>
<td>26587</td>
</tr>
</tbody>
</table>

**Table 6** Concrete material properties according to TS 500 code for the BPS

<table>
<thead>
<tr>
<th></th>
<th>compressive str. of concrete (MPa)</th>
<th>tensile strength of concrete (MPa)</th>
<th>modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>case-1</td>
<td>26</td>
<td>1.78</td>
<td>30572</td>
</tr>
<tr>
<td>case-2</td>
<td>28</td>
<td>1.85</td>
<td>31197</td>
</tr>
<tr>
<td>case-3</td>
<td>30</td>
<td>1.92</td>
<td>31800</td>
</tr>
<tr>
<td>case-4</td>
<td>32</td>
<td>1.98</td>
<td>32384</td>
</tr>
</tbody>
</table>

**Fig. 9** General layout of the tested BPS (Schoettler et al. 2015)

**Fig. 10** Selected maximum peak displacement points for comparison in the time history graphs of the BPS
To compare top displacement values obtained from the experimental and numerical results of the BPS, four maximum peak points in the displacement time history graphs for each case were selected (Fig. 10). Schoettler et al. (2015) indicated that after the 3rd earthquake acceleration loading, significant cracking developed at the base of the bridge pier, and spalling at pier concrete occurred (Fig. 11). Severe damage regions at the base of the bridge pier were obtained from all numerical analysis results. No difference between experimental and all numerical results in terms of damaged regions and categorized damage ratio was observed.

### 3.2.1. Investigation of the BPS according to ACI-318 code

Top story displacement time history graphs of numerical and experimental results for Case-1 to Case-4 according to ACI-318 are seen in Fig. 12. Mean differences between displacements obtained from experimental and the Case-1, Case-2, Case-3, Case-4 numerical solutions were calculated as 4.2%, 3.2%, 2.4%, and 3.0% by using Eq. (4), respectively. The minimum difference was obtained for the Case-3 solution. The comparisons of the experimental test results with the numerical analysis results obtained according to ACI-318 code provisions were given in Table 7. The differences in compressive strength and elasticity modulus of concrete are determined as 0.0% and 7.2%, respectively. It was seen that the result of Case 3 has the best agreement with the PEER test results.

| Table 7 Comparison of material properties obtained according to ACI-318 for the BPS |
|---------------------------------|-----------------|-----------------|
|                                | compressive strength | elasticity modulus |
|                                | (MPa)              | (MPa)            |
| Numerical analysis             | 30.0              | 25743.0          |
| Laboratory test results        | 30.0              | 24000.0          |
| Difference                     | 0.0%              | 7.2%             |
3.2.2. Investigation of the BPS according to TS 500 code

Top displacement time history graphs of experimental, and numerical results obtained according to the TS 500 code are seen in Fig. 13. Mean differences between displacements obtained from the experiment and the Case-1, Case-2, Case-3, and Case-4, numerical solutions were calculated as 3.9%, 3.3%, 4.6%, and 5.5% by using Eq. (4), respectively. The minimum difference was obtained in the Case-2 solution.
Table 8 Comparison of material properties obtained according to TS 500 for the BPS

<table>
<thead>
<tr>
<th></th>
<th>compressive strength (MPa)</th>
<th>elasticity modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Numerical analysis</td>
<td>28</td>
<td>31197</td>
</tr>
<tr>
<td>Laboratory test results</td>
<td>30</td>
<td>24000</td>
</tr>
<tr>
<td>Difference</td>
<td>6.7 %</td>
<td>29.9 %</td>
</tr>
</tbody>
</table>

The comparisons of the numerical analysis results obtained using TS 500 code provisions with the experimental test results were given in Table 8. The differences in compressive strength and elasticity modulus of concrete are determined as 6.7% and 29.9%, respectively. It was seen that the result of Case 2 has the best agreement with the PEER test results.

4 CONCLUSION

In this study, the effectiveness of ACI-318 and TS 500 design codes in numerical modeling of the seismic responses of RC structures is investigated. For this purpose, the seismic responses of two RC structures experimented in the laboratory are used. The first structure is a two-dimensional three-bay and four-story RC bare frame structure that was tested in European Laboratory for Structural Assessment. The second structure is an RC tall bridge pier that was tested at Pacific Earthquake Engineering Research Center. The numerical solutions are obtained by using the Force-based Fiber Element Approach. The concrete material properties for the RC structural elements are defined according to ACI-318 and TS 500 design codes. The steel material properties are determined by considering experimental test results. The experimental and numerical analysis results are compared with regard to top displacements and damage zones. The concrete material properties which provide the best approximation to the experimental results are researched for both design codes. For the Force-based Fiber Element Approach, obtained results from this investigation can be summarized as follows:

- In the numerical solution of the Bare frame building results, the differences between the experimental and numerical analysis results obtained according to ACI-318 are computed as 1.84%, and 0.92% for compressive strength and elasticity modulus of the concrete, respectively. However, the difference is computed as 17.10% for the tensile strength of concrete.

- In the Bare frame building results obtained according to TS 500, the difference for compressive strength is obtained as 1.84%. However, the differences in tensile strength and elasticity modulus are obtained as 26.3% and 42.3%, respectively.
• In the Bridge pier structure results, the differences between experimental and numerical analysis results obtained according to ACI-318 for compressive strength and elasticity modulus of concrete are determined as 0.0% and 7.2%, respectively.

• The differences in the Bridge pier structure between experimental and numerical analysis results obtained according to TS 500 are computed as 6.7% for compressive strength, and 29.9% for elasticity modulus of concrete.

• Numerical results obtained from ACI-318 code results are more approximate to the experimental results than the TS500 code results for the Force-based Fiber Element Approach with regard to displacement response and damage regions.

• In the future, the investigations can be extended for the other design codes and different equations can be suggested for the concrete material properties of RC structures.

DECLARATIONS

Compliance with Ethical Standards

Ethical Approval
The manuscript does not include any human subjects and animals, and also this study does not involve any case reports/case series.

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Conflicts of interest
The authors declare that they have no conflicts of interest relevant to the content of this article.

Informed Consent
This study does not involve any human or animal participants. Therefore, there is no informed consent for the submitted paper.

Authorship contributions
All authors have participated in (a) conception and design, or analysis and interpretation of the data, (b) drafting the article or revising it critically for important intellectual content, and (c) approval of the final version.

Availability of Data and Material
Available upon request
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