Extensive damages or collapse observed for silos subjected to earthquake excitations can lead to significant monetary and environmental losses while human life may also be set at high risk. Therefore, the design of earthquake-resistant silos is essential to safeguard silos’ integrity and functionality over their lifetime. In case of adopting the performance-based design approach, well-defined damage limit states are required to quantify the expected seismic failures of silos. Nevertheless, to the authors’ best knowledge, only marginal research effort has been already spent to quantify appropriate damage limit states for silos. To this end, the current study is dedicated to determine damage states for a typical cylindrical reinforced concrete silo, containing granular material with the use of a multi-record, non-linear incremental dynamic analysis (IDA). The numerical model of the silo was developed in ABAQUS. The advanced hypoplastic constitutive model, extended with the intergranular strain approach, was adopted to simulate the dynamic performance of the granular material while the silo wall was modelled by using the concrete damaged plasticity model. Additionally, granular material-structure interaction was considered through a Coulomb friction interface. Both static and dynamic (i.e., time history) analyses were performed to validate both the granular material-structure interaction and the silo’s dynamic response. Furthermore, IDA curves, in terms of base shear over top displacement, were calculated with the use of an appropriately selected and scaled set of earthquake strong ground motions. The IDA curves were eventually used to quantify five damage limit states according to maximum top displacement criterion.

Keywords: reinforced concrete silo; hypoplasticity; concrete damaged plasticity; incremental dynamic analysis; damage limit states

1. INTRODUCTION

Silos are common industrial facilities, designed to contain valuable materials (e.g., cement, sand, grain). The undisturbed operation of silos during their lifetime is of high importance for the relevant stakeholders and the society, since silos’ damages or collapse can lead to excessive direct and indirect monetary losses being associated with structural failures and the wastage of the contained material. In case that toxic or pollutant materials are stored by the silos, environmental failures can emerge that may also set human life at risk. Silos are constructed worldwide and they are exposed to natural perils that can jeopardize their structural integrity and reliability. Several occasions have been already documented when silos suffered from severe structural failures or even collapsed after past seismic events (Bechtoula, 2005; Dogangun et al., 2009; Tremblay et al., 2013). As reported by Tremblay et al. (2013), two primary types of structural failures governed the seismic-induced damages of reinforced concrete (RC) silos due to an earthquake in Chile (2010). Especially, the RC silos experienced flexural cracking that led to severe cracking damage for silos of short-to-moderate height without stiffeners around the base openings. Flexural-shear failure was also observed for RC silos, most likely initiated by the crushing of the concrete at the transition from thinner (at higher elevation) to thicker walls (closer to the base) respectively. Apart from such a discontinuity in walls’ cross section, the use
of concrete of poor quality with low compressive strength and the design of the cross sections with reinforcement less than the minimum code requirements were also found relevant for structural failures (Tremblay et al., 2013). A detailed description of severe damages that a silo complex experienced due to the Zemmouri earthquake (Algeria, 2003) is provided by Bechtoula (2005). More specifically, the most heavily damaged rc silo battery, found to be almost full with grain by the time of the earthquake event, suffered from severe concrete crushing, rebar fractures and buckling as well as partial sliding of the external concrete shell was observed.

The detrimental effects that earthquake ground excitations may have on silos’ functionality and structural integrity have already triggered significant research effort. Along these lines, Wagner & Meskouris (2001) focused on the seismic performance of a silo by undertaking numerical investigation through a model that accounts for the granular material, simulated therein by a hypoplastic material, while frictional contact interface was considered between the filling material and the rc walls. The latter were modelled through shell finite elements. Elastic numerical models of rc silos with varying slenderness ratio have been also used to elaborate their seismic response by performing time history analyses with the use, though, of quite few strong ground motions (Durmuş & Livaoglu 2015; Livaoglu & Durmuş 2015; 2016). Numerical models of steel silos subjected to earthquake motions were also used to study the effect of the granular material – structure interaction on the silos’ behavior (Nateghi & Yakhchalıyan 2011) while the EN 1998-4 (CEN, 2006) approach to calculate the design loads imposed by the dynamic response of the filling material, was comparatively assessed with results from nonlinear time history analysis (Nateghi & Yakhchalıyan 2012; Holler & Meskouris 2006).

Moreover, the hypoplastic constitutive law was adopted by Jagtap et al. (2015) to model the dynamic response of granular materials of varying density and their influence on the seismic performance of steel silo was evaluated accounting for the lateral displacements along the height of the structure. Based on the discussion made above, several advancements have been already made regarding the detailed modelling of the complex, seismic response of silos filled with granular material. However, the limited number of earthquake records, widely adopted by the aforementioned studies in order to represent the seismic actions, may undermine the reliability of the calculated response results since the seismic motions have been already found to affect significantly the variability in the structural behavior (Elnashai & McClure, 1996). On the other hand, a systematic evaluation of the seismic vulnerability of silos has only been recently undertaken by Guo et al. (2016), who performed incremental dynamic analysis (IDA) (Vamvatsikos & Cornell, 2002) to calculate fragility curves for a rc silo exposed to a suite of 10 code-compatible earthquake motions. However, an arbitrarily chosen IDA curve, being associated with a single seismic motion, was eventually adopted to determine the damage states restricting, in such a way, their reliability and applicability in relevant cases. Moreover, the special configuration of silos (i.e., tall and hollow structures) along with the complex dynamic interaction between the contained material and the walls render problematic and questionable the use of damage states that were defined for similar structures, like high-rise buildings or chimneys.

Along these lines, the rather marginal research effort that has been spent to identity damage states for silos triggered this study, which addresses the reliable quantification of rc silos’ damage states on the basis of a multi-record IDA. The latter allows calculating dynamic capacity curves (i.e., IDA curves) by processing the seismic response results from a large number of time-history analyses performed with the use of 12 earthquake records, appropriately selected and scaled to cover a broad range of structural behavior from purely elastic to fully inelastic response. A numerical model was developed accounting for non-linear constitutive laws both for the rc structural members (i.e., silo’s wall) and the filling granular material, which was modelled on the basis of the hypoplasticity theory refined by intergranular strain approach. The numerical model was validated to ensure dynamic response results with increased accuracy. The damage states were defined according to a top displacement criterion that can be readily used within the robust framework of the performance-based design.

2. NUMERICAL MODEL OF SILO

A typical, cylindrical rc silo of $h = 15$ m height, diameter (internal), $d$, equal to 10 m and wall thickness, $t = 0.2$ m was numerically investigated by the use of commercial software ABAQUS (Simulia, 2013). To reduce the significant computational time that is commonly associated with time
history analysis of large structures subjected to seismic excitations, half of the silo was modelled taking advantage of its symmetrical configuration. Moreover, the silo wall was modelled as constrained to the flat base, which, in turn, was considered to have fixed boundary conditions. Four node shell elements with reduced integration (S4R) were used to model the wall and the base while the granular material was simulated by the use of three dimensional eight node solid elements with full integration (C3D8) to avoid hourglassing. Additionally, the hypoplastic model for sand, initially proposed by von Wolfersdorff (1996) for cyclic loading, was used herein to numerically reproduce the behavior of a granular material contained in the silo. Especially, hypoplasticity, being diverse from the classical elastoplasticity that requires the distinction between elastic and plastic deformations as well as the definition of the yield surface, was defined herein on the basis of eight material constants, which are related to geometric and material properties of grains and can be determined from soil standard laboratory tests. The necessary parameters for the hypoplastic material model are defined in the following: $\varphi$, is the critical angle of friction, $e_{i0}$, $e_{d0}$ and $e_{ed0}$ are the critical, maximum and minimum void ratios, $b$, is the granular hardness, $n$ controls the curvature of the oedometric compression curve while $\alpha$ and $\beta$ are exponents related to the soil shear stiffness and the peak friction angle, respectively. Despite the advanced representation of the granular material response that the hypoplasticity provides, the plain hypoplastic model is unable to simulate small load amplitudes in case of cycling loading and hence, the deformations accumulate unrealistically causing, in turn, the so-called ratcheting effect (Wagner & Meskouris, 2000). To waive this limitation, a new state variable defined as intergranular strain has been introduced and it can be understood as a macroscopic measure of micro-deformations of the interface layer between grains. In this manner, not only the deformation of the granular skeleton due to grain rearrangement is considered, but also the deformation of the interface layer (Herle & Gudehus, 1999; Niemunis & Herle, 1997). However, the intergranular strain extension of the hypoplastic model requires the definition of five additional parameters: $R$ is the maximum intergranular strain and defines the size of the elastic range, $m_1$ is a parameter controlling the initial shear modulus upon 180° strain path reversal and in the initial loading, $m_8$ governs the initial shear modulus upon 90° strain path reversal, while $\chi$ and $\beta$ are material constant exponents related to the transition between different deformation modes. With regards to applications in numerical models subjected to dynamic loading, the intergranular strain extension of hypoplasticity has been found suitable and accurate (Wagner & Meskouris, 2000). The hypoplasticity, refined herein by the intergranular strain approach, was introduced to ABAQUS (Simulia, 2013) through a user-defined material subroutine (UMAT), originally developed by Gudehus et al. (2008). The silo was assumed to contain Hochstetten sand and the corresponding material parameters used to define the hypoplastic material are listed in Table 1. The interaction between structure and filling material is modelled as a Coulumb friction interface with a coefficient of friction, $\mu = 0.4$. The material is prevented to penetrate the wall by introducing hard contact behavior.

Concrete category of C25/30 (i.e., compressive strength equal to 25 N/mm²) was considered herein while the concrete damaged plasticity (CDP) model, already embedded in the ABAQUS material library, was adopted to simulate the cyclic performance of the concrete structural members subjected to dynamic excitation. According to the CDP model, two failure mechanisms are considered, namely tensile cracking and compressive crushing, which are described by the damage variables, $d_t$ and $d_c$, respectively, being dependent on the plastic strain. The damage variables represent the degradation of the elastic stiffness and range from zero to one representing no damage and total loss of strength respectively. The stress-strain curves and the related damage variables of the CDP model are plotted in Figs. 1 and 2 while the material parameters, needed to define the specific material model are defined in the following: $E$ is the elastic modulus of the concrete material, $\nu$ is the Poisson’s ratio, $\beta$ is the dilation angle, $\epsilon$ is the flow potential eccentricity, $\sigma_{b0}/\sigma_{0}$ is the ratio of biaxial and uniaxial compressive yield strengths, $K$ is the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield, and $\mu_v$ is the viscosity parameter (Simulia, 2013). Table 2 lists the values adopted herein for the aforementioned parameters of the CDP model. Regarding the concrete reinforcement, B400 steel bars were considered and a bilinear material model (yield strength $f_y = 400$ N/mm², ultimate tensile strain $\epsilon_{u} = 14\%$) was used to reinforce the silo’s wall both longitudinally and transversely with ratios, $\rho$, equal to 1.01% (Ø16/200) and 0.77% (Ø14/200) respectively. The latter are complied with the EN 1992-1 provisions (CEN, 2008). Furthermore, Rayleigh material damping was applied for both concrete and granular material with mass and
stiffness proportional damping factors respectively ($\alpha_R$ and $\beta_R$) selected appropriately in order to reach critical damping ratio, $\zeta$, equal to 1% within the frequency range of interest.

Table 1. Hypoplastic material parameters of Hochstetten sand (Niemunis & Herle, 1997)

<table>
<thead>
<tr>
<th>$\rho$</th>
<th>$\varphi_c$</th>
<th>$h_s$</th>
<th>$n$</th>
<th>$e_{d0}$</th>
<th>$e_{e0}$</th>
<th>$\alpha$</th>
<th>$B$</th>
<th>$R$</th>
<th>$m_R$</th>
<th>$m_T$</th>
<th>$\beta_r$</th>
<th>$\chi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1700 kg/m$^3$</td>
<td>33°</td>
<td>1.5e6 kPa</td>
<td>0.28</td>
<td>0.55</td>
<td>0.95</td>
<td>1.05</td>
<td>0.25</td>
<td>1.5</td>
<td>0.0001</td>
<td>5</td>
<td>2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2. Concrete damaged plasticity material parameters (McCrum et al., 2016; Nateghi & Yakhchalian, 2011)

<table>
<thead>
<tr>
<th>$E$</th>
<th>$\nu$</th>
<th>$\rho$</th>
<th>$\beta$</th>
<th>$\epsilon$</th>
<th>$\sigma_{in}/\sigma_{e0}$</th>
<th>$K$</th>
<th>$\mu_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>29 GPa</td>
<td>0.19</td>
<td>2400 kg/m$^3$</td>
<td>25°</td>
<td>0.1</td>
<td>1.16</td>
<td>0.667</td>
<td>$10^{-7}$</td>
</tr>
</tbody>
</table>

3. VALIDATION OF THE NUMERICAL MODEL

3.1 Validation based on static response

The granular material-structure interaction is crucial for the numerical model and may affect the response results; hence, thorough validation scheme is necessary to be undertaken. Along these lines, the numerical static analysis results concerning the distribution of the vertical and horizontal pressures along the silo’s depth were compared with the Jansen’s analytical solution that considers frictional contact between the filling material and the silo. Especially, the vertical, $P_v$, and horizontal, $P_h$, pressure respectively, exerted by the granular material on the silo wall, is formulated on the basis of Janssen (1895) as follows:
where \( g \) is the gravitational acceleration. The numerical analysis was carried out in accordance with the modelling principles presented above. However, the geometry of the silo was slightly modified for the static validation purposes; the silo’s height was increased to 20 m enabling the detailed comparison of the numerical analysis results with the theoretical approach for a taller structure with increased flexibility. The concrete structure was assumed to behave in the elastic regime while the hypoplasticity was considered to model the response of the granular material with properties identical to the ones presented by Table 1. Furthermore, the mobilized angle of friction, computed by ABAQUS, was used to calculate the pressure distribution according to the analytical expressions.

\[
P_v = \frac{\rho gd}{4\mu} \left[ 1 + \sin \varphi_c \left( 1 - \exp \left( \frac{-4\mu h (1 - \sin \varphi_c)}{d (1 + \sin \varphi_c)} \right) \right) \right] \quad \text{and} \quad P_h = \frac{1 - \sin \varphi_c}{1 + \sin \varphi_c} \rho g d
\]

(1)

Based on Fig. 3, both the horizontal and vertical pressures, calculated along the depth of the silo on the basis of the numerical analysis, are in close agreement with the pressures’ distribution derived from Janssen’s theory. Therefore, the validation of the numerical model, in terms of the static response, is considered to be sufficient. It is only adjacent to the silo’s base, where slightly different pressure results were calculated by the two methods, being attributed to their difference regarding the modelling of the silo’s base. Especially, Janssen’s analytical formulation disregards the base configuration and its effect on the pressures formation. On the other hand, the numerical investigation accounts for the silo’s base while friction interaction was considered between the granular material and the base.

### 3.2 Validation based on dynamic response

The numerical model developed herein for the rc silo and the granular material was subjected to an artificially generated seismic excitation and the horizontal pressure on the silo’s wall obtained from the time history analysis, were compared to the dynamic pressure, \( \Delta_{ph,s} \), calculated on the basis of EN 1998-4 provisions:

\[
\Delta_{ph,s} = \Delta_{ph,so} \cos \theta
\]

(2)

where \( \Delta_{ph,so} \) is the reference pressure and \( \theta \) is the angle between the radial line to the point of interest on the wall and the direction of the horizontal component of the seismic action (CEN, 2006). At points on the silo’s wall at a vertical distance \( x \) from a flat bottom or the apex of a conical or pyramidal hopper, the reference pressure, \( \Delta_{ph,so} \), may be calculated as:

\[
\Delta_{ph,so} = \alpha(z) \min\left(r^*; \frac{3}{2} x\right) \quad \text{and} \quad r^* = \min\left(h_b; \frac{d_e}{2}\right)
\]

(3)

where \( \alpha(z) \) is the ratio of the response acceleration of the silo at a vertical distance \( z \) from the
equivalent surface of the stored contents, to the acceleration of gravity while \( \gamma \) is the bulk unit weight of the particulate material. As it concerns the definition of the factor \( r^* \), \( h_b \) is the overall height of the silo, from a flat bottom or the hopper outlet to the equivalent surface of the stored contents and \( d_c \) is the inside dimension of the silo parallel to the horizontal component of the seismic action (CEN, 2006). Regarding the calculation of the \( \alpha (z) \) term, no variation was considered along the structure's height while the 5% damped spectral acceleration of the artificially generated accelerogram at the silo's fundamental period, calculated via eigenvalue analysis equal to 0.19 s, was found to be 0.6 g.

The conservativeness, commonly related to code prescriptions, is reflected by the results plotted in Fig. 4, for which the distribution along the structure’s height of the dynamic pressure was calculated on the basis of the Eurocode 8-based approach and the time history analysis results. The latter were obtained accounting for specific time instants, when the dynamic pressure reaches its maximum (Fig. 4, left), or considering the envelope of maxima values along the silo’s height (Fig. 4, right). The calculation of higher dynamic pressure according to the code-based approach, corroborated also elsewhere for almost the entire silo’s height (Jagtap et al., 2015; Nateghi & Yakhchalian, 2012), was found to be valid irrespectively of the sides of the silo investigated herein. Despite the mild deviations discussed above regarding the dynamic pressure calculations, such a comparison can still be used to provide a concrete indication of the validity of the numerical model developed herein that favors its further use in the current study.

![Graph](image)

Figure 4. Artificially generated accelerogram used for the validation scheme of the numerical model (left); Dynamic pressure distribution along the silo’s height at time instant \( t \) (left); envelope of dynamic pressure maxima along the silo’s height (right)

4. SELECTION OF EARTHQUAKE STRONG GROUND MOTIONS

The validation scheme, already presented above both in terms of static and dynamic response, reinforced the suitability of the numerical model to predict with increased accuracy the silo’s seismic response, being, in turn, necessary to calculate the damage states through the performance of IDA. The latter requires the use of earthquake strong ground motions, appropriately selected to maintain the natural variability of the seismic excitations and hence, lead to reliable response results. Along these lines, a suite of 12 earthquake motions, recorded during past seismic events and obtained from the PEER-NGA Database (Chiou et al., 2008), was formed on the basis of criteria that account for: the earthquake moment magnitude \( (M_w) \) – severe earthquake events of \( M_w > 6 \) were considered; the epicentral distance \( (R) \) – far field motions of \( R > 20 \) km were chosen; soil conditions at the recording station’s site – the selected motions were recorded from very dense soil profile or stiff soil that correspond to C and D site categories based on NEHRP provisions (Building Seismic Safety Council, 2003). The basic characteristics of the selected 12 seismic motions are listed in Table 3.
<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake event</th>
<th>$M_w$</th>
<th>$R$ (km)</th>
<th>Recording station</th>
<th>pga (g)</th>
<th>Site class</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Northridge, USA (17.01.1994)</td>
<td>6.69</td>
<td>31.6</td>
<td>Big Tujunga</td>
<td>0.245</td>
<td>C</td>
</tr>
<tr>
<td>2</td>
<td>San Fernando, USA (09.02.1971)</td>
<td>6.61</td>
<td>39.5</td>
<td>LA-Hollywood Stor FF</td>
<td>0.174</td>
<td>D</td>
</tr>
<tr>
<td>3</td>
<td>Coalinga, USA (02.05.1983)</td>
<td>6.36</td>
<td>30.1</td>
<td>Cantua Creek School</td>
<td>0.227</td>
<td>D</td>
</tr>
<tr>
<td>4</td>
<td>Big Bear, USA (28.06.1992)</td>
<td>6.46</td>
<td>40.5</td>
<td>Desert Hot Springs</td>
<td>0.225</td>
<td>D</td>
</tr>
<tr>
<td>5</td>
<td>Smart, Taiwan (21.09.1983)</td>
<td>6.32</td>
<td>65.5</td>
<td>SMART1 E01</td>
<td>0.203</td>
<td>D</td>
</tr>
<tr>
<td>6</td>
<td>Cape Mendocino, USA (25.04.1992)</td>
<td>7.01</td>
<td>53.3</td>
<td>Eureka-Myrtle &amp; West</td>
<td>0.178</td>
<td>D</td>
</tr>
<tr>
<td>7</td>
<td>San Fernando, USA (09.02.1971)</td>
<td>6.61</td>
<td>20.0</td>
<td>Lake Hughes #12</td>
<td>0.366</td>
<td>C</td>
</tr>
<tr>
<td>8</td>
<td>Edgecumbe, New Zealand (02.03.1987)</td>
<td>6.60</td>
<td>24.2</td>
<td>Matahina Dam</td>
<td>0.256</td>
<td>C</td>
</tr>
<tr>
<td>9</td>
<td>Landers, USA (28.06.1992)</td>
<td>7.28</td>
<td>86.0</td>
<td>Yermo Fire Station</td>
<td>0.245</td>
<td>D</td>
</tr>
<tr>
<td>10</td>
<td>Borrego Mtn, USA (09.04.1968)</td>
<td>6.63</td>
<td>70.8</td>
<td>El Centro Array #9</td>
<td>0.130</td>
<td>D</td>
</tr>
<tr>
<td>11</td>
<td>Victoria, Mexico (09.06.1980)</td>
<td>6.33</td>
<td>36.7</td>
<td>Chihuahua</td>
<td>0.150</td>
<td>D</td>
</tr>
<tr>
<td>12</td>
<td>Kobe, Japan (17.01.1995)</td>
<td>6.90</td>
<td>19.3</td>
<td>Port Island (0 m)</td>
<td>0.315</td>
<td>D</td>
</tr>
</tbody>
</table>

*p* pga is used herein for peak ground acceleration.

5. IDA ANALYSIS AND RESPONSE RESULTS

The numerical model including the rc silo and the granular material was subjected to the selected motions and the multi-component IDA performed herein was used to derive the dynamic capacity curves (or else IDA curves) that were defined in terms of absolute maxima values of the base shear, $V$, and top displacement, $\delta$, of the silo wall. It is notable that the $V-\delta$ values, being the output of the time history analysis, do not necessarily occur at the same time instant. However, the use of such absolute maxima values provide a safety upper bound in the structural response (Papanikolaou & Elnashai, 2005) and consequently, in the estimated damage states. An adaptive strategy for strong ground motion scaling was also applied in order to facilitate the efficient performance of the IDA scheme that would not compromise the reliability of the calculated response with the burden associated with the increased computational time. Particularly, the selected accelerograms were scaled to four basic levels of pga equal to 0.25 g, 0.5 g, 1.0 g and 2.0 g. Then, the dynamic response results, obtained by the excitation of numerical model using the initially scaled motions, were examined and if necessary, the scaling factors were populated so as the performance of additional time history analyses safeguarded the calculation of the silo’s seismic response within the entire range of the structural behavior.

![Figure 5. IDA curves of the rc silo model (left). Median IDA curve (blue), its derivative (red), and the defined damage limit states (grey) (right).](Image)
In total, 118 time history analyses were performed by using ABAQUS and the 12 IDA curves were calculated on the basis of 10 (on average) pairs of response parameters $V - \delta$. According to Fig. 5 (left), an initial purely elastic region can be clearly detected for the entire set of the IDA curves until the first yielding occurs. The latter is typical to any structural configuration modelled with elements that disregard initial cracking (Vamvatsikos & Cornell, 2002). The first “elastic” slope is followed by a second hardening phase that ends at the final yielding zone, whereafter the post-yield (plateau-like) response area is reached. As non-linear behavior takes over after the initial linear elastic region, higher variability is observed among the curves, mainly attributed to the inherent accelerogram-specific nature of the IDA (Vamvatsikos & Cornell, 2002). Indeed, the structural systems have been widely found to experience dissimilar responses when excited by various seismic motions.

6. DEFINITION OF DAMAGE STATES

Structural damage varies from none to complete damage as a continuous function of deformations. However for practical reasons, generalized states are commonly used to describe both qualitatively and quantitatively the extension of the damage that the structural systems experience due to hazardous sources. For the current study, the widely known damage state definitions of HAZUS (2003), provided though for building configurations, are adopted: None, Slight; Moderate; Extensive; and Complete. These limit states were quantified herein on the basis of peak displacement, $\delta$, at the top of structure corresponding to the onset of damages of varying severity. In order to define the displacement limits for the damage states based on silo’s seismic performance within an average sense, the median IDA curve was calculated along with its derivative function (Fig. 5, right). The latter, reflecting the degradation of the lateral stiffness of the structure in relation with the top displacement, was introduced by Vargas et al. (2013) to quantify damage states for rc building configurations. Such a generic approach enables its use for any kind of structures that experience degrading performance during the seismic loading. Therefore, the first significant drop of the lateral stiffness occurred for $\delta = 0.3$ cm, reflects the transition from the purely elastic (undamaged) state to the one associated with slight damage. The next significant reduction of the silo’s lateral stiffness (i.e., approximately 45%) was calculated for a displacement level equal to $\delta = 2.5$ cm, considered herein as the threshold between the Slight and the Moderate damage state. Based on Fig. 5 (right), the silo seems to exhaust its yielding zone for $\delta = 5.0$ cm, where the drop of lateral stiffness was calculated equal to 22%. The latter limit is considered to differentiate the Moderate from the Extensive damage state, while the post-yield (plateau-like) zone of the silo’s global performance extends up to $\delta = 20$ cm, where the lateral stiffness is reduced to a negative value when reaching the descending part of the median IDA curve. Table 4 lists the damage states determined herein by providing both qualitative and quantitative description. Moreover, the following sections provide the necessary reinforcement for the decisions made above regarding the damage states based on results directly from the IDA of the rc silo.

Table 4. Qualitative and quantitative description of damage states.

<table>
<thead>
<tr>
<th>Damage state</th>
<th>Qualitative description</th>
<th>$\delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>No damages are observed; the structure remains entirely in an elastic state.</td>
<td>0 – 0.3 cm</td>
</tr>
<tr>
<td>Slight</td>
<td>Flexural or shear type hairline cracks occur on the wall surface.</td>
<td>0.3 – 2.5 cm</td>
</tr>
<tr>
<td>Moderate</td>
<td>Incipient concrete spalling is observed. Repair might be required.</td>
<td>2.5 – 5.0 cm</td>
</tr>
<tr>
<td>Extensive</td>
<td>Extensive spalling takes place. The structure exceeds its yield capacity.</td>
<td>5.0 – 20.0 cm</td>
</tr>
<tr>
<td>Complete</td>
<td>Structure has collapsed or is in imminent danger of collapse.</td>
<td>20.0 cm</td>
</tr>
</tbody>
</table>

5.1 None damage

The structure remains entirely undamaged through the initial, linear elastic part of the median IDA curve (Fig. 5, right) ranging, in terms of top lateral displacement up to 0.3 cm that is considered to correspond to the first yielding of the structure, i.e., the initiation of tensile cracking. The latter is corroborated by the results of the IDA and especially the contour plots (Fig. 6) of the tensile cracking
failure-related damage variable, $d_t$, of the ABAQUS CDP model. Depending on the seismic record (ID) and its amplitude ($pga$), either limited or none indication of tensile damages can be observed in Fig. 6 and the corresponding displacement values were found to range around the threshold of $\delta = 0.3$ cm, adopted herein to differentiate the None and Slight damage state respectively.

**5.2 Slight damage**

In the Slight damage state, the silo wall typically sustains tensile forces enabling in such a way the reinforcement to be utilized. The significant expansion of the tensile damages is depicted by Fig. 7, where the tensile cracking failure-related damage variable, $d_t$, has nearly reached its maximum for the majority of the cross-sections found between the base of the silo and the 2/3 of its height. The silo’s top displacement that indicates, in average terms, the prevailing tensile damages was found herein to be equal to 2.5 cm. At the same time, the latter corresponds, on average, to the onset of the compression damages for the concrete material as can be seen by Fig. 8, in which low values for the compressive crushing failure-related damage variable, $d_c$, can be detected for the cross-sections of the silo’s wall close to the base. The adoption of the specific displacement threshold to differentiate the Slight and Moderate damage states is further corroborated by the corresponding reduction in the lateral stiffness (Fig. 5, right), being the result of the initiated damages.

**Figure 6.** Contour plots of the tensile cracking failure-related damage variable, $d_t$, calculated for the rc silo that its performance corresponds approximately to the limit between the None and Slight damage states.

**Figure 7.** Contour plots of the tensile cracking failure-related damage variable, $d_t$, calculated for the rc silo that its performance corresponds approximately to the limit between the Slight and Moderate damage states.

**Figure 8.** Contour plots of the compressive crushing failure-related damage variable, $d_c$, calculated for the rc silo that its performance corresponds approximately to the limit between the Slight and Moderate damage states.
5.3 Moderate damage

Regarding the Moderate damage state, the already initiated compression damages are considered to remain non-severe and localized to the lower elevation of the silo’s wall. In fact, the onset of compression damages, corresponding to $\delta = 2.5$ cm ($\S$5.2), does not result in concrete spalling that eventually takes place, as can be seen by Fig. 9, at later stage of the silo’s dynamic response ranging, in terms of the top silo’s displacement, between 2.5 cm up to 5 cm. At the same time, the compressive strain for the concrete material reaches 0.4% corresponding to incipient spalling (Priestley, 1998). The top silo’s displacement of 5 cm, chosen herein to differentiate the Moderate and the Extensive damage states, corresponds to additional drop in the silo’s lateral stiffness (Fig. 5, right), indicating the transition from final yielding to the plateau-like, post-yield section of the median IDA curve.

5.4 Extensive damage

The silo’s wall sustains severe concrete spalling that propagates gradually from the base to higher elevation of the wall. The severe concrete spalling is quantified herein by the high value of the compression damage variable, i.e., $d_c = 0.950$, that, in turn, corresponds to compressive (crushing) strain equal to 1.44% (Fig. 1, right). A slightly higher compressive strain has been already suggested by Kowalsky (2000) as a threshold for unrepairable spalling, therefore $\varepsilon_c = 1.44\%$ is considered to be an adequate strain limit for severe concrete spalling that was found herein to correspond, in average terms, to silo’s top displacement equal to 20 cm (Fig. 10). The latter displacement, related also to an additional slight decrease in the silo’s lateral stiffness (Fig. 5, right), was adopted as the threshold between the Extensive and the Complete damage states respectively.

5.5 Complete damage

The Complete damage indicates that the structure is in imminent danger of collapse or has already collapsed. Such an extensively degraded capacity of the silo is corroborated by the lateral stiffness, which takes a negative value as reaching the descending part of the median IDA curve within this ultimate stage of the structural response (Fig. 5, right).
6. CONCLUSION

Within the framework of the current study, the widely adopted IDA was conducted to quantify the seismic damage limit states for a typical cylindrical rc silo containing granular material. To this end, a refined finite element model was developed that considers non-linear constitutive material laws both for the rc structural members (i.e., the silo’s walls) and the sandy filling material. The latter was modelled on the basis of the advanced hypoplasticity theory that was further refined by the intergranular strain approach enabling, in such a way, to capture with increased accuracy the complex dynamic behaviour of the sandy material and its interaction with the walls of the silo when subjected to seismic base excitations. To safeguard the reliability of the structural analysis scheme followed herein and its response results, the finite element model was successfully validated in terms of both static and dynamic performance. The IDA was facilitated by a set of 12 seismic records selected appropriately to maintain the inherent variability of the earthquake strong ground motions. The capacity curves (or else IDA curves), calculated herein in terms of absolute maxima values of the base shear, \( V \), and top displacement, \( \delta \), of the silo wall, represent the silo’s seismic response within the entire range of the structural behaviour. Therefore, the median capacity curve and its derivative, being the silo’s lateral stiffness, were used to define quantitatively five damage limit states and the associated top displacement thresholds. Such a quantification of the seismic damage, reinforced herein by IDA response results in terms of both tensile cracking- and compressive crushing-related damage indicators, serves the basis to undertake the performance-based design of silos. The latter is considered nowadays as a robust design tool to meet successfully high performance objectives for structure and infrastructure systems of increased importance for the relevant stakeholders and the society.

7. REFERENCES


