INVESTIGATION OF THE POST-SHEAR FAILURE BEHAVIOUR OF REINFORCED CONCRETE PILES IN THE GRONINGEN AREA

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ABSTRACT

The Dutch Practical Guideline, NEN-NPR9998: December 2015, describes methods to assess whether existing buildings are able to withstand earthquakes which may occur in the Groningen region. It has often not been possible to demonstrate that existing piles will be able to continue to support buildings subjected to NPR defined seismic demands with pile capacities exceeded by as much as an order of magnitude, meaning that they cannot be relied upon to provide a credible vertical load path. Reports of post-earthquake evaluations in other seismic regions indicate that, even when piles severely damaged, they often continue to provide gravity support to buildings, such that collapse does not occur. This anecdotal experience cannot be directly extrapolated to the Groningen region because most of the types of piles in Groningen lack seismic detailing which is used tectonic seismic regions. The disruption and cost involved in retrofitting building foundations to prevent collapse is very high and the purpose of this study was to numerically determine if existing pile systems have reserve vertical load carrying capacity beyond conventional code based force limits for reinforced concrete sections such that resources can be used wisely. The study has shown that the soil, pile, pile-cap and ground beam system of existing buildings, even if not originally seismically detailed, may often have hidden reserves to be able to continue carrying vertical loads bringing potentially very significant economies to the assessment of existing building to the near collapse limit state of the NEN NPR 9998.

Keywords: Groningen; reinforced concrete; post-shear failure; soil-structure interaction; LS-DYNA.

1. INTRODUCTION

Piles are a popular foundation in the Netherlands for transferring superstructure gravity loads to the soil. Based on statistical data it can be estimated that about 40% of residential buildings in the region have foundation piles and the majority of these piles comprised of either reinforced concrete piles or timber piles with precast reinforced concrete caps. Induced-earthquakes have been observed in the north of the Netherlands due to the extraction of gas from the Groningen gas field, resulting in the development of the current seismic assessment guideline (NEN-NPR 9998:December 2015) for Groningen and subsequent seismic building assessments. Seismic assessments for individual buildings that have been performed to date in accordance with the NEN-NPR 9998 indicate that, for the design earthquake demand, the shear and/or the rotational capacity of existing piles are often exceeded at the near collapse limit state. However what the codes and design guidelines do not address is whether or not piles that have their capacity momentarily exceeded can continue to support buildings, such that the near collapse limit state is not exceeded.

This paper will present a summary of an investigation into the post failure behavior of the lightly reinforced concrete piles encountered in the Groningen area and includes:

- Details of the most common reinforced concrete pile types used in the Groningen area and examples of pile capacity exceedance assessed with the seismic demand according to NEN-NPR 9998, for the Near Collapse Limit State;
- A summary of published laboratory tests which consider post-shear failure behaviour of reinforced concrete columns, which are then used to calibrate the analytical method adopted for the numerical simulations of the typical Groningen piles; and

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The results of numerical analyses to assess the post failure behaviour of typical lightly reinforced concrete piles in the Groningen area and an assessment of the piles ability to provide a credible vertical load path after failure.

This work was carried out based on the NEN NPR9998: December 2015 version of the hazard definition which has greater demands than the latest, although still draft, hazard definition as per the June 2017 version of NEN NPR9998. This means that the displacement demands are likely to be reduced compared to the demands applied to the piles as part of this study.

2. SUMMARY OF REINFORCED CONCRETE PILES IN THE GRONINGEN AREA

Reinforced concrete piles of existing buildings in the Groningen region typically comprise either continuous flight auger (CFA), driven precast (PC) and driven precast pre-stressed (PS) piles. Typical detailing of these piles are shown in Figure 1 and Table 1 outlines the material and geometric properties of the selected pile sample.

![Figure 1. Detailed of typical piles in the Groningen area, CFA (left), PC (middle) and PS (right)](image)

Table 1 Material and geometric properties of the selected reinforced concrete pile sample

<table>
<thead>
<tr>
<th>Reference</th>
<th>Pile Type</th>
<th>Pile Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFA-A02</td>
<td>Continuous flight auger</td>
<td>Ø350mm; C20/25; B500B; 5Ø10; hoops Ø8-750</td>
</tr>
<tr>
<td>PC-B02</td>
<td>Driven precast</td>
<td>290x290mm; C35/45; FeB220; 4Ø12; stirrups Ø6-150</td>
</tr>
<tr>
<td>PS-C02</td>
<td>Driven precast, pre-stressed</td>
<td>250x250mm; C35/45; FeP1860; 4Ø9.3; end spirals</td>
</tr>
</tbody>
</table>

With the NEN-NPR 9998:2015 seismic demand, the code-based methods suggest that the existing pile stock in Groningen is likely to fail in a large portion of the area of interest. NEN-NPR 9998 refers to NEN-EN1992-2 for estimating shear capacity of existing reinforced elements, though for the assessment of buildings for the near collapse limit state, partial factors of 1.0 are applied as well as best estimate material properties, rather than characteristic values. Table 2 outlines some of the pile failures that have been identified during seismic assessments on a range of existing building typologies with different pile types.
Table 2. Example of assessed pile shear failures at surface

<table>
<thead>
<tr>
<th>Type of building</th>
<th>Pile Ref.</th>
<th>Analysis method</th>
<th>Shear demand</th>
<th>Shear capacity</th>
<th>Failure criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential mid rise</td>
<td>CFA</td>
<td>NLTHA</td>
<td>164kN</td>
<td>108kN</td>
<td>Capacity &lt; Demand</td>
</tr>
<tr>
<td>Hospitality low rise</td>
<td>PS</td>
<td>RSA</td>
<td>136kN</td>
<td>73kN</td>
<td>Capacity &lt; Demand</td>
</tr>
<tr>
<td>Residential low rise</td>
<td>PC</td>
<td>NLTHA</td>
<td>65kN</td>
<td>44kN</td>
<td>Capacity &lt; Demand</td>
</tr>
</tbody>
</table>

Results of pseudo-static analyses with linear-elastic precast concrete piles (250 x 250mm) of a building in the Groningen area, which were subjected to kinematic loads only, demonstrated an exceedance of both their shear and bending capacity at soil depth. Further analyses undertaken in LS-DYNA using non-linear moment-curvature pile properties indicated that the plastic rotations exceeded, by almost a factor of two, the collapse prevention limit state acceptance criteria of ASCE 41-13 (2014).

3. EMPIRICAL METHOD AND LABORATORY TESTS TO ASSESS POST-SHEAR FAILURE BEHAVIOUR OF REINFORCED CONCRETE COLUMNS

An extensive literature review was carried out to identify existing knowledge that could contribute to achieving the objectives of this study. Only a few studies were found that specifically looked at the shear failure behaviour of reinforced concrete piles with light shear reinforcement. Studies and research programs dealing with pile failures in shear are typically more focused on soil-structure interaction than on the shear capacity of the pile and post-shear failure behaviour. However, there is extensive research that has investigated the shear failure behaviour of reinforced concrete columns in moment frames and bridge piers subjected to horizontal drifts.

To investigate alternative assessment methodologies for the pre and post-shear failure behaviour of reinforced concrete piles, it was decided to focus primarily on the analogy with reinforced concrete columns and to initially ignore any effects of soil confinement when calibrating the pile component models. This is most likely a valid initial assumption for the near surface part of the pile, given the relatively low soil confining pressures on piles at this depth in the typical soil profiles of the Groningen region.

3.1 Empirical Method

Moehle and Elwood (2005) developed an empirical model to predict the ultimate drift displacement of reinforced concrete columns failing in shear or combined flexure/shear at the onset of axial failure. Figure 2 provides a graph of the estimated column drifts at axial failure versus the axial load, as calculated with this empirical model for the Groningen pile sample group. The term “maximum capacity model” in the figure corresponds to conditions where only the shear friction contribution is considered.

![Figure 2](image.jpg)

Figure 2. Estimated ultimate drifts at axial failure for pile sections PC-B02, CFA-A02 and PS-C02.

This model predicts that for the CFA and for the PC piles, ultimate drifts in excess of 1.5% can be achieved
without a loss of vertical bearing capacity for the expected axial loads. However, the PS piles seem to sustain a very low level of ultimate drifts at axial failure. This empirical model was calibrated based on piles with shear reinforcement ratios between 0.07% and 0.18%, whereas the typical Groningen piles often have lower, if any, shear reinforcement. Given the poor predicted performance of the PS piles, these were not analysed further due to their expected low maximum drift performance compared with the estimated seismic demands.

3.2 Laboratory Tests

The laboratory test references shown in Table 3 were used to calibrate the 3D non-linear LS-DYNA analysis models for square and circular sections. These tests were selected among a large number of tests in the literature because of their close resemblance to the reinforced concrete specifications of the “Groningen” pile sample group. The tests from Lynn et al. (2001) were used in the calibration of the empirical model referenced in 3.1 while the tests from Ranf R.T. and Jaradat O.A. were originally performed to determine the shear capacity of bridge piers and therefore did not extend beyond shear failure, nevertheless they were the only relevant lab tests comparable to CFA piles.

<table>
<thead>
<tr>
<th>Experiment Reference</th>
<th>Author</th>
<th>Year</th>
<th>Notes on type of laboratory tests</th>
<th>Document Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>2CMH18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C3R</td>
<td>Ranf R.T.</td>
<td>2006</td>
<td>Circular columns, cantilevering from fixed floor restraint. Displacements applied at the top.</td>
<td>(Ranf et al. 2006).</td>
</tr>
<tr>
<td>S1</td>
<td>Jaradat O.A.</td>
<td>1996</td>
<td>Circular columns scaled from original bridge piers, cantilevering from fixed floor restraint. Displacements applied at the top.</td>
<td>(Jaradat, O.A.1996)</td>
</tr>
</tbody>
</table>

3.3 Post-Axial Failure Behaviour

After the onset of initial axial failure there is very limited research applicable to this study that would help to evaluate the residual axial capacity-axial degradation post failure. Nevertheless, there are some indications, refer to Elwood and Moehle (2008), to suggest that after failure a reinforced concrete pile could still retain a proportion of its original axial capacity. Furthermore, the axial load demand on the pile and the ability of the above structure to redistribute the vertical and horizontal loads to alternative load paths should play a critical role in the evaluation of the residual axial capacity of the pile post failure. For a gravity column, if redistribution of vertical load to adjoining columns does not occur, axial failure represents a failure mechanism with high probability of resulting in building collapse. For piles however, the axial capacity degradation could potentially result in redistribution of vertical loads to bearing of foundation beams and pile caps on the soil and/or to adjoining piles and be less damaging for the super-structure.

4. DESCRIPTION AND CALIBRATION OF MODELLING TECHNIQUE

Before investigating the behaviour of “Groningen” type piles, work was undertaken to calibrate the analytical modelling approach against available test data. The following paragraphs summarise this calibration work.

4.1 Model materials, restraints and loading

The analysis model was initially tested on the simulation of a laboratory test of RC columns described in (Lynn et al. 2001). The test involved a constant axial load (controlled by actuators B & C) and a varying lateral displacement (A) applied to the test specimen, as shown in Figure 3.
The analysis models featured a fully detailed 3-D non-linear representation of the reinforcement and concrete using constraints to simulate the interaction between the rebar and the concrete. An isometric view of the finite element model (FEM) representation of the test specimen is shown in Figure 3.

The concrete behaviour was modelled using the non-linear smeared crack Winfrith material model developed by Broadhouse and Neilson (1987) which has been extensively validated against experiments. The elements used were 3D solids with a single integration point and with a maximum mesh size of 20mm. The model accounts for the increase in strength due to confinement, but does not modify the maximum strain limit of the material. Erosion was added to the concrete elements in the cover to enable concrete cover to fall away.

The reinforcement was modelled with 1D beam elements with 2x2 integration points. It was connected to the concrete using the constraint formulations within LS-DYNA. Bond slip was implemented assuming good bond properties as taken from the fib Model Code (2010).

The base of the test piece was fully-fixed at every node. The top of the test piece was coated in rigid shell elements that were rotationally restrained and therefore kept horizontal. Out-of-plane translations were restrained and the top was free to translate vertically.

The applied vertical load includes both an axial (vertical force as per lab test specification) and gravity. The vertical loads were ramped over a period of 0.1s and allowed to settle prior to the application of lateral displacement demands. The vertical load was applied as a constant nodal load and held through the analysis.

Cyclic lateral displacement demands were applied slowly to the top of the model to mimic the original quasi-static laboratory test conditions.

4.2 Model Reference Output

The reference analyses output described in Table 4 was used for both the calibration with the laboratory tests and later on for the evaluation of the pile components performance. The references are typically used for most of the laboratory tests on reinforced concrete column in moment frames discussed in the previous sections and they typically constitute the reference values used to generate backbone curves. For the analyses output it is assumed that axial failure starts after 5mm vertical shortening of the component measured from the top beam.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Description (Δy, Δs, Δa are reported as values based on cycle displacement amplitude)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δy</td>
<td>mm</td>
<td>Lateral displacement at first longitudinal reinforcement yielding</td>
</tr>
<tr>
<td>Δs</td>
<td>mm</td>
<td>Lateral displacement when the peak shear resistance has dropped by 20%</td>
</tr>
<tr>
<td>Δa</td>
<td>mm</td>
<td>Lateral displacement when column has shortened by 5mm</td>
</tr>
<tr>
<td>Δcf</td>
<td>mm</td>
<td>Lateral displacement at collapse or at end of time history</td>
</tr>
<tr>
<td>V_u</td>
<td>kN</td>
<td>Peak shear resistance</td>
</tr>
<tr>
<td>M_{V_u}</td>
<td>kNm</td>
<td>Bending moment at V_u</td>
</tr>
</tbody>
</table>
Symbol | Unit | Description (Δy, Δa, Δaf are reported as values based on cycle displacement amplitude)
---|---|---

The laboratory records, where available, identified the displacement of the component at axial failure without specifying the amount of vertical shortening associated with the failure. For the laboratory tests, and when the value was provided, Δa and Δaf are given the same value.

### 4.3 Results from Calibration Runs

Table 5 shows a series of comparisons between the results of LS-DYNA simulations and the original laboratory tests based on the reference analysis output values defined in Table 4. Not all reference output values were available for all laboratory tests used, where those values were not provided the table below shows “n/a” on the corresponding ratio of accuracy.

Table 5 Comparison between LS-DYNA simulations and original laboratory test results for reference output values based on level of accuracy ratios (analytical simulation result over lab test value).

<table>
<thead>
<tr>
<th>Test label</th>
<th>Member sizes</th>
<th>Accuracy ratios based on reference output values</th>
<th>( \Delta y )</th>
<th>( \Delta a )</th>
<th>( \Delta af )</th>
<th>( \Delta a + \Delta af )/2</th>
<th>( V_u )</th>
<th>( M_{Vu} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2CLH18</td>
<td>457mm square</td>
<td>0.97</td>
<td>0.70</td>
<td>1.38</td>
<td>1.04</td>
<td>0.94</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>2CMH19</td>
<td>457mm square</td>
<td>0.94</td>
<td>1.07</td>
<td>1.07</td>
<td>1.07</td>
<td>0.73</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>254mm round</td>
<td>0.81</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>0.85</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td>C3R</td>
<td>505mm round</td>
<td>0.92</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>1.00</td>
<td>0.97</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4 provides more detail of the analytical results for the simulations of 2CLH18 in terms of shear force at the top of the column (black line), applied displacements (red line) and vertical shortening of the top beam (green) for a square section. The damaged state of the specimen at the onset of collapse is also provided at the right side of Figure 4.

Figure 4. LS-DYNA results for 2CLH18.

Based on the results of this calibration the following can be concluded:

- The simulations provide a relatively good prediction of the maximum shear capacity recorded in the lab tests and the associated bending moment. Departures appear to become more relevant as axial load on the specimen increases but predictions are on the conservative side.
- The analytical simulations predict the deformation at first yield of longitudinal reinforcement and at shear failure quite accurately even though for the lab test with low axial load (2CLH18) the performance associated with sustained shear capacity between peak shear resistance and Δa for increased displacements is underestimated. This however does not affect the alignment of the calibration with the axial failure behaviour of the original laboratory test results.
- Axial failure and post axial failure behaviour seems to align with the preliminary indication obtained from simplified methods. The amount of axial load appears to drive the speed at which the specimen collapses after failure. 2CMH18 shows axial collapse almost immediately after axial failure (>5mm shortening), while 2CLH18 shows a prolonged shortening which could suggest a relatively slow deterioration of axial load capacity.

5. COMPONENT SIMULATION FOR TYPICAL GRONINGEN PILES

The pile sections for CFA and PC piles were modelled and analysed with the same component settings used for the laboratory tests simulations with a set height of 2946 mm between fixed restraints and applied displacements of incrementing amplitude in cycles of three for each drift amplitude. Model variations based on axial load, length of component between fixed restraints, reinforcement quantities and concrete strength were analysed for PC-B02 and CFA-A02. Seven different levels of axial load, four longitudinal and four transverse reinforcement amounts, two effective pile lengths and four concrete strength variations giving a total of 48 analyses were performed (see Table 6). Variations studies listed in the rows of Table 6 where performed while keeping all other properties of the selected reference pile unchanged.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Percentage variation</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-75</td>
<td>-50</td>
<td>-25</td>
<td>Original</td>
<td>+25</td>
<td>+50</td>
<td>+100</td>
</tr>
<tr>
<td>Axial load (P)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Effective length (L_{eff})</td>
<td>✓</td>
<td></td>
<td></td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength (f'_c)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal reinf. (As)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse reinforcement (At)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results of these simulations in terms of reference output drift values are summarized in the charts included in Figure 5 for axial load and concrete strength variations.

![Figure 5](image)

**Figure 5** Displacement reference output values in % drifts for PC-B02 (top) and CFA-A02 (bottom) shown for axial load variations (left) and concrete strength variations (right).
The impact of the variations analysed on the axial failure behaviour of the component’s performance can be observed in Figure 6 and Figure 7. Vertical shortening of the column is plotted in the time domain with the cycles of applied horizontal drifts. The applied displacements terminated at an amplitude of 117.8mm for all models to have comparable results and to keep the analytical processing time within reasonable limits.

The following observations were made on the variations study carried out:

- **Axial load variations**: axial shortening of the columns proceeds gradually after axial failure for values of axial load less than \(385.0 \text{kN} \times 0.14 \frac{f'_c A_g}{A} \) for CFA-A02 and \(420.5 \text{kN} \times 0.12 \frac{f'_c A_g}{A} \) for PC-B02. Maximum shear capacity is reached for all specimens analysed within the first three displacement cycles and this, for components with an axial load less than the above reference values, represents a relatively small portion of the total displacement capacity of the components analysed before their collapse under the constantly applied axial load.

- **Concrete strength variations**: Increasing the strength of the concrete provides a greater displacement post-axial failure, identified by increasing drift values between \(\Delta_a\) and \(\Delta_{af}\). Reducing concrete strength has the opposite effects and failure occurs for smaller drifts.

- **Length variations**: the relative drift difference between \(\Delta_a\) and \(\Delta_{af}\) does not change much between the variations analysed suggesting that the vertical shortening would typically progress with the same speed independently from the length between fixed restraints.

- **Longitudinal and transverse reinforcement variations**: Reinforcement has been increased and reduced based on equivalent bar area (spacing was not adjusted). The results show that reinforcement variations, within the already very low pile reinforcement ratios, have little impact on the post-axial failure behaviour.
6. PILE SIMULATION BASED ON A PROJECT REFERENCE DEMAND

The analytical modelling method described in Section 4 was then tested to analyse piles using the seismic demands derived from the assessment of a terraced house in Groningen Figure 9. The assessment of the building was carried out using Nonlinear Response History Analyses (NLRHA) with Dynamic Soil-Structure Interaction (DSSI). The earthquake loading demands that have been applied are as per the requirements of Annex F7 of NEN-NPR9998:December 2015 using appropriate time history scaling factors, as specified in the NPR.

The time histories of velocities (relative to the pile’s toe) and axial load demands on the top of the pile were extracted from the original model of the building with as-built foundations but an upgraded superstructure. The assessment of the model originally concluded that piles would fail at the connection with the grade beams and at the peat layers located a few meters from the surface very early in the ground motion. By assuming the piles, where the codified shear capacity was exceeded, could no longer sustain vertical loads, the foundation beams were considered insufficient to provide a vertical load path to support the building. The explicit pile component model included the soil block around the pile in order to simulate soil constraints and pile bearing. This would also allow direct bearing of the capping beam on the soil if the pile would fail under applied axial load. This simulation focused predominately on the near surface part of the pile where inertia forces typically govern the seismic demand. At this stage, kinematic effects due to the soil displacement were not taken into consideration. The pile head time history demands were extracted from the full model assuming elastic-plastic pile behaviour in order to prevent premature failure and generate full time history results for the component model.

Figure 8 presents an overview of the building’s foundations, details of the precast pile selected (edge pile of the foundation grillage) for this simulation and the assumed soil profile. Figure 9 shows the analytical component used for the simulation of the piles.

![Diagram of building foundations and pile simulation model]

Figure 8 Foundation layout al location of piles for component simulations (left), pile details (middle) and soil profile (right).
6.1 Assumptions on Component Modelling

In the absence of a full model for the super-structure, it is assumed that the reinforced concrete capping beam at the top of the pile is rotationally restrained out of plane at both ends. This is in line with the assumptions made on the previous analyses on pile components and lab tests discussed in this report. Beam flexibility, if considered critical, could be re-evaluated in the overall model once the component behaviour has been established. However, this is not the case according to the results obtained from the analysis of the terraced house. Axial load extracted from the pile top section of the full model was applied to the top of the beam of the component model to simulate vertical load from above the pile and to allow for vertical load path distribution between pile and beam if the pile experiences axial failure. A 50 mm gap was provided between the underside of the capping beam and the soil, which represents typical as-built conditions where consolidation of the soil has occurred. Finer meshing of soil elements was provided around the pile and below the foundation beam to better capture soil strain and stress distribution.

6.2 Results of the simulation on edge pile

The results of the analysis of the pile component are shown in Figure 10 and Figure 11. For Figure 10 the pile damage state is shown prior to shear failure, at shear failure and after shear failure with the capping beam becoming in contact with the soil.
The explicit component model predicts that the edge pile fails in shear and undergoes shear deterioration and axial shortening, as generally predicted in the pile simulations presented in Section 5. In addition, this model develops a supplementary vertical load path through soil bearing of the capping beam, providing a reduction of axial demand on the pile. This behaviour appears to delay the axial capacity deterioration of the element. At the end of the time history, the pile still contributes to supporting around 50% of the original applied gravity load with a total vertical deformation of the capping beam of 80 mm (50 mm of physical gap plus 30 mm of soil deformation, see Figure 11).

Figure 11 Time history results: full model with perfectly-plastic piles (top) and explicit pile component model (bottom), where beam vertical shortening (green), pile shear resultant (black), pile vertical load (blue and dashed blue for total applied axial load) and beam bearing reaction on soil (light brown, only for pile component).

7. CONCLUSIONS

Based on the work carried out in this phase of the study, the following conclusions can be made for Continuous Flight Auger (CFA) and driven precast (PC) reinforced concrete piles:

- The level of axial force demand on piles is one of the main parameters affecting their post-axial failure performance. In the component analyses with cyclic loading and for axial loads greater than $0.15A_g \cdot f'_c$ (where $A_g$ is the gross section area and $f'_c$ is the mean compressive strength of concrete), the axial failure is typically followed quickly by the collapse of the component when analysed as a free standing column. Piles with lower axial force levels can achieve greater lateral displacement capacities with progressive shortening in the damaged part of the element whilst maintaining some vertical load carrying capacity.

- The test carried out with seismic demand extracted from an existing assessment provides evidence that residual axial capacity on piles post-shear failure could be justified based on the pile vertical shortening concentrating in the damaged portion of the element. This shortening behaviour, without complete loss of axial capacity of the pile, may allow the capping beams to come into contact with the soil to develop a supplementary vertical load path which further relieves the pile from applied axial load. In this case, vertical support to the building could be provided by a combination of post-shear failure residual axial capacity of the piles and bearing of foundation beams on the soil.

- Validation through physical testing is recommended for the calibrated modelling technique which
was presented in this study. The post-shear failure axial capacity of reinforced concrete piles in Groningen, including pile performance at depth where soil confinement may provide beneficial effects, is also recommended for further study.

- A likely deformation and load carrying capacity of piles, with pile caps and ground beams, surrounded by soil, is usually neglected when following code based methods. The development of “backbone curves” for typical Groningen piled foundations in common soil profiles would enable more realistic seismic assessments to the near collapse limit state.

The findings from this study may bring considerable benefits to the seismic assessment of existing buildings in the Groningen region and may minimise the need for foundation strengthening. The hazard definition in the draft NPR 9998 June 2017 has changed and it is anticipated that the seismic deformation demands are generally expected to be reduced compared to the December 2015 NPR9998 hazard definition.

8. ACKNOWLEDGEMENTS

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9. REFERENCES


