Quasi-static cyclic tests and plastic hinge analysis of RC structural walls

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**A B S T R A C T**

This paper presents the results of quasi-static cyclic tests on six reinforced concrete (RC) walls performed at the ETH Zurich. These large-scale tests investigate the effect of different vertical reinforcement contents and different reinforcement ductility properties typical for Central Europe on the deformation behaviour of slender RC walls. The test data documenting the global and local behaviour of the test units is available online and can therefore serve as a reference point for the research community. The experimental results show the importance of the reinforcement content and the ductility properties of both the boundary and web reinforcement for the deformation behaviour of the walls. By comparing base curvatures derived from experimental data with curvatures obtained from section analysis, strain limits characterising different limit states in plastic hinge analysis are suggested. These strain limits can be used in the performance-based design and assessment of RC structural walls.

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1. Introduction

Reinforced concrete (RC) walls are structural elements frequently used in buildings to provide lateral stiffness and strength against wind and earthquake actions. In regions of moderate and high seismicity such walls are typically designed and detailed according to capacity-design principles [1]. Nevertheless, significant structural differences can be observed: In regions with a high seismic hazard, walls with purely rectangular cross sections are seldom used because the strength and ductility required against strong earthquake actions is better achieved by walls with boundary elements. If, as an exception, walls with a rectangular cross section are used, they often need large reinforcement ratios. Therefore, in countries with high seismicity, research and experimental testing focus on RC structural walls featuring these characteristics (e.g. [2–5]).

On the contrary, in regions of moderate seismicity, such as e.g. Switzerland, wall sections are typically purely rectangular and often a total reinforcement ratio of less than 1% is sufficient to provide the required lateral stiffness and strength to the structure. Moreover, in most countries of Central Europe the number of storeys of office and factory buildings is limited and therefore the axial load ratio of the structural walls is relatively low.

Important differences exist also with respect to the mechanical properties of the reinforcement. Particularly in Central Europe the importance of the ductility properties of the longitudinal reinforcement on the displacement capacity of the walls was for a long time undervalued and reinforcing steel was merely rated for its strength rather than its deformation capacity. As a consequence a portion of the existing RC wall buildings was constructed with reinforcing steel possessing inferior ductility properties.

For these reasons, an experimental programme was conducted at the ETH Zurich to investigate the effects of the particularities of structural walls typical for countries of moderate seismicity in Central Europe on the seismic behaviour of such walls. This paper presents quasi-static cyclic tests on six RC walls that were carried out within this programme. After this brief introduction Section 2 describes the test units and the test setup. Section 3 continues with brief descriptions of the hysteretic behaviour, failure mechanisms and crack patterns of the six walls. In Section 4 selected test results are presented and discussed focusing on the local deformation behaviour of the walls since it controls the failure of the wall. The test results illustrate the detrimental effects of low reinforcement contents and inferior ductility properties of the longitudinal reinforcement on the displacement capacity of structural walls in general but also reveal that the mechanical properties of the web reinforcement should receive the same attention as the boundary reinforcement when aiming for a ductile behaviour of the walls. Finally, Section 5 discusses the application of the plastic hinge method when assessing the inelastic deformation behaviour of the test units. A detailed description of the experiments and a compilation of all test data in digital format are available online [6].

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Table 1
Summary of the test unit properties.

<table>
<thead>
<tr>
<th>Test unit</th>
<th>Sectional forces at the base</th>
<th>Reinforcement ratios</th>
<th>Stabilising reinf.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$N$ (kN)</td>
<td>$N/A_{f_{c}}$</td>
<td>$V/0.8l_{w}b_{w}$ (MPa)</td>
</tr>
<tr>
<td>WSH1</td>
<td>689±3</td>
<td>0.051</td>
<td>1.40</td>
</tr>
<tr>
<td>WSH2</td>
<td>691±4</td>
<td>0.057</td>
<td>1.50</td>
</tr>
<tr>
<td>WSH3</td>
<td>686±7</td>
<td>0.058</td>
<td>1.89</td>
</tr>
<tr>
<td>WSH4</td>
<td>695±6</td>
<td>0.057</td>
<td>1.85</td>
</tr>
<tr>
<td>WSH5</td>
<td>1474±29</td>
<td>0.128</td>
<td>1.83</td>
</tr>
<tr>
<td>WSH6</td>
<td>1476±6</td>
<td>0.108</td>
<td>2.49</td>
</tr>
</tbody>
</table>

- Standard deviation of the applied axial load over the duration of the test.
- Ratios computed with largest sectional forces measured during the test.

2. Experimental investigation

The test series comprised six large-scale RC cantilever walls tested under quasi-static cyclic loading. In the following, the geometry of the test units, the material properties, the test setup, the instrumentation, and the loading history are described.

2.1. Test units

The test units were half-scale models of the lower part of a RC wall in a six-storey reference building with a total height of 20.4 m [7]. The height of the test units was determined considering the reduction of the shear span due to higher mode effects. The geometry of the test units was based on the smallest shear span for which the entire flexural capacity was reached and this shear span corresponded to 45% of the total wall height. All test units had identical dimensions and shear reinforcement which satisfied capacity design principles but was governed by minimum horizontal reinforcement requirements. The test units differed regarding the longitudinal reinforcement layout and content, the ductility properties of the reinforcement, the confining and stabilising reinforcement, and the applied axial load. The test units were labelled WSH1 to WSH6. The test units were 2.00 m long and 0.15 m wide. The length of the shear span $l_{v}$ of the test units was 4.56 m for WSH1 to WSH5 and 4.52 m for WSH6; this corresponds to shear span ratios of 2.28 and 2.26, respectively. Fig. 1 shows the cross-sections of the reinforcement layouts in the potential plastic zone for all six test units. Fig. 2 shows as an example the side view of the reinforcement cage of WSH3. The axial force, shear force and bending moment ratios as well as the reinforcement ratios and information on the stabilising reinforcement of the six test units are summarised in Table 1.

Fig. 2. Vertical reinforcement layout of Test Unit WSH3. All dimensions in mm.

2.2. Material properties

The strength and ductility properties of the reinforcing steel are given in Table 2 and a typical stress–strain-relationship
for a D12 reinforcing bar is displayed in Fig. 3. All values are mean values obtained from uniaxial tension tests on reinforcing bars with a clear length of 750 mm. The ductility properties are characterised by the hardening ratio $R_{h}/R_{02}$ and the total elongation at maximum force $A_{pt}$, i.e. the ultimate strain. The tensile strength $R_{m}$ and the proof stress at 0.2% non-proportional elongation $R_{02}$ were obtained from the material tests dividing the force of the testing machine by the nominal area of the bar. $R_{02}$ is used as an approximation of the yield strength $f_y$ since none of the bars used for the construction of the test units exhibited a pronounced yield plateau [8]. The ultimate strain $A_{pt}$ was determined as the stroke of the testing machine at maximum force divided by the clear length of the test specimen while also accounting for the elastic deformation of the testing machine. Note that $A_{pt}$ can vary considerably between different test methods. The test method employed here aimed at deriving an ultimate strain measure which is comparable to the strain capacity obtained by the manual method according to EC2 [9]. In this method the strain capacity of a reinforcing bar is determined as the plastic strain of a 100 mm gauge length – not including the part of the specimen that fractured – plus the elastic strain at maximum load. Computing the strain as average strain over the rather long clear length of 750 mm serves as an approximation of the strain capacity of the bars outside their region of rupture. In addition to the stroke measurements, extensometer measurements with a gauge length of 300 mm were carried out. Measurements on bars where the fracture occurred outside the extensometer gauge length showed that $A_{pt}$ obtained from the extensometer readings was about 8% smaller than $A_{pt}$ obtained from the stroke. In the last two columns of Table 2 the reinforcing bars are classified according to EC2 [9] based on the hardening ratio $R_{h}/R_{02}$ and $A_{pt}$. The mean hardening ratios of the longitudinal and shear reinforcing bars varied between 1.03 and 1.28 while their strain capacity ranged between 2.34% and 7.85%.

Each test unit was cast in two phases: In the first phase the foundation block and the wall up to a height of 1.5 m above the foundation block were cast upright. For the second phase the test unit was laid down horizontally in order to simplify the construction. The mechanical properties of the concrete given in Table 3 belong to the samples taken during the first phase of the casting. The density $\rho_c$ and the cube strength $f_{cu}$ were obtained from cube specimens with a 200 mm edge length while the cylinder strength $f'_c$ and the modulus of elasticity $E_c$ were obtained from cylinder specimens with a diameter of 150 mm and a height of 300 mm. Further details on the material tests are given in [6].

### Table 2

Mean values and standard deviations of mechanical properties of the reinforcing steel.

<table>
<thead>
<tr>
<th>$D_{nom}$ (mm)</th>
<th>$R_{02}$ (MPa)</th>
<th>$R_{m}$ (MPa)</th>
<th>$R_{m}/R_{02}$ (–)</th>
<th>$A_{pt}$ (%)</th>
<th>Classification $R_{h}/R_{02}$ – $A_{pt}$</th>
<th>Class$^d$ (–)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (6x)$^a$</td>
<td>547.3 ± 6.1</td>
<td>619.9 ± 7.1</td>
<td>1.13 ± 0.01</td>
<td>4.64 ± 0.16</td>
<td>$M^a$ - $L^c$</td>
<td>A</td>
</tr>
<tr>
<td>6 (6x)</td>
<td>583.6 ± 5.0</td>
<td>600.7 ± 8.2</td>
<td>1.03 ± 0.01</td>
<td>2.34 ± 0.87</td>
<td>$L$ - $P$</td>
<td>&lt; A</td>
</tr>
<tr>
<td>3.5 (3x)</td>
<td>655.9 ± 29.7</td>
<td>662.0 ± 28.0</td>
<td>1.02 ± 0.00</td>
<td>0.60 ± 0.04</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>WSH1</td>
<td>10 (12x)</td>
<td>583.1 ± 4.4</td>
<td>747.4 ± 3.9</td>
<td>1.28 ± 0.01</td>
<td>$H$ - $H$</td>
<td>C</td>
</tr>
<tr>
<td>6 (10x)</td>
<td>484.9 ± 10.2</td>
<td>534.5 ± 8.7</td>
<td>1.10 ± 0.01</td>
<td>5.76 ± 0.50</td>
<td>$M$ - $M$</td>
<td>B</td>
</tr>
<tr>
<td>4.2 (3x)</td>
<td>526.0 ± 6.0</td>
<td>583.2 ± 5.2</td>
<td>1.11 ± 0.00</td>
<td>3.30 ± 0.28</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>WSH2</td>
<td>12 (6x)</td>
<td>601.0 ± 6.3</td>
<td>725 ± 2.1</td>
<td>1.21 ± 0.01</td>
<td>$H$ - $H$</td>
<td>C</td>
</tr>
<tr>
<td>8 (4x)</td>
<td>569.2 ± 4.0</td>
<td>700.2 ± 3.3</td>
<td>1.23 ± 0.01</td>
<td>7.34 ± 0.29</td>
<td>$H$ - $M$</td>
<td>B (~C)</td>
</tr>
<tr>
<td>6 (6x)</td>
<td>489.0 ± 4.3</td>
<td>552.2 ± 3.3</td>
<td>1.13 ± 0.00</td>
<td>6.45 ± 0.33</td>
<td>$M$ - $M$</td>
<td>B</td>
</tr>
<tr>
<td>4.2 (6x)</td>
<td>562.2 ± 1.8</td>
<td>615.0 ± 3.0</td>
<td>1.09 ± 0.00</td>
<td>3.06 ± 0.66</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>WSH3</td>
<td>12 (6x)</td>
<td>576.0 ± 2.6</td>
<td>674.9 ± 1.8</td>
<td>1.17 ± 0.01</td>
<td>$H$ - $M$</td>
<td>B (~C)</td>
</tr>
<tr>
<td>8 (6x)</td>
<td>583.7 ± 5.5</td>
<td>714.4 ± 5.1</td>
<td>1.22 ± 0.01</td>
<td>7.85 ± 0.66</td>
<td>$H$ - $H$</td>
<td>C</td>
</tr>
<tr>
<td>6 (6x)</td>
<td>518.9 ± 13.8</td>
<td>558.7 ± 6.7</td>
<td>1.08 ± 0.02</td>
<td>5.45 ± 0.41</td>
<td>$M$ - $M$</td>
<td>B</td>
</tr>
<tr>
<td>4.2 (6x)</td>
<td>562.2 ± 1.8</td>
<td>615.0 ± 3.0</td>
<td>1.09 ± 0.00</td>
<td>3.06 ± 0.66</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

### Table 3

Mechanical properties of the concrete.

<table>
<thead>
<tr>
<th>Test unit</th>
<th>$\rho_c$ (kg/m$^3$)</th>
<th>$f_{cu}$ (MPa)</th>
<th>$f'_c$ (MPa)</th>
<th>$E_c$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSH1</td>
<td>2397 ± 28</td>
<td>55.3 ± 1.8</td>
<td>45.0 ± 2.1</td>
<td>44.4 ± 5.1</td>
</tr>
<tr>
<td>WSH2</td>
<td>2421 ± 23</td>
<td>55.0 ± 2.1</td>
<td>40.5 ± 2.8</td>
<td>37.1 ± 0.1</td>
</tr>
<tr>
<td>WSH3</td>
<td>2381 ± 18</td>
<td>56.8 ± 1.6</td>
<td>39.2 ± 2.1</td>
<td>35.2 ± 1.5</td>
</tr>
<tr>
<td>WSH4</td>
<td>2378 ± 15</td>
<td>58.8 ± 1.7</td>
<td>40.9 ± 1.8</td>
<td>38.5 ± 2.0</td>
</tr>
<tr>
<td>WSH5</td>
<td>2404 ± 19</td>
<td>56.0 ± 2.3</td>
<td>38.3 ± 1.4</td>
<td>36.1 ± 1.5</td>
</tr>
<tr>
<td>WSH6</td>
<td>2383 ± 22</td>
<td>59.2 ± 2.9</td>
<td>45.6 ± 0.3</td>
<td>36.9 ± 0.7</td>
</tr>
</tbody>
</table>

$^a$ Number of bar samples tested.  
$b$ $R_{m}/R_{02}$: Poor < 1.05 < Low < 1.08 < Medium < 1.15 < High.  
$c$ $A_{pt}$: Poor < 2.5% < Low < 5% < Medium < 7.5% < High.  
$d$ According to EC2, Annex C1 [9].

### Fig. 3

Typical stress–strain relationship and ductility properties for D12 bars used within the tests.

### 2.3. Test setup, instrumentation, and loading history

A photo of the test setup is shown in Fig. 4. The test units were clamped onto the strong floor by large diameter post-tensioning bars. At the wall head the test units were subjected to a horizontal displacement history applied by two servo-controlled hydraulic actuators with a force capacity of ±500 kN and a displacement capacity of ±100 mm each. The two actuators were connected in series since the upper bound estimate of the displacement capacity...
of the units computed prior to the testing was larger than the displacement capacity of a single actuator. For WSH6 only one ±1000 kN actuator with a displacement capacity of ±100 mm was used. The actuators were fastened to the wall head by means of twelve post-tensioning bars which ensured a tight contact of the actuators to the wall head throughout the test.

For Test Units WSH1 and WSH2 the axial load was applied by an unbonded post-tensioning tendon which was placed in a metal duct embedded in the test unit along its centreline. The tendon had a dead-end anchor at the base of the foundation and a live anchor at the wall head. The tendon was post-tensioned by a hollow core jack placed on top of the test unit. This system was very practical but too expensive for all six test units since a new tendon and base anchor would have been required for each of the tests. For the remaining four test units the axial load was therefore applied by a tendon on either side of the test unit which was anchored against the strong floor and post-tensioned by flat jacks placed on top of the test unit. In both axial load systems the jacks were connected to a load-follower that kept the axial load nearly constant throughout the entire test (see Column 2 in Table 1).

During the testing, the behaviour of the walls was monitored by recording a large number of hard-wired measurements and by performing manual measurements after selected load steps. The hard-wired instruments are shown in Figs. 5(a) and (b). The measured quantities were local and global deformations as well as the applied forces. The local deformations that were measured comprised linear variable differential transformer (LVDT) measurements of the wall edge elongations, diagonal LVDT measurements for determining the shear deformations, and strains along the corner bars in the foundation.

In addition, Demec measurements (Whitmore gauge measurements) and crack width measurements were taken manually. For each test unit ten major cracks were chosen at the end of the first two cycles and the width of each of these cracks was measured at five locations. Demec measurements were taken on the East and West face of the test unit (Figs. 5(c) and (d)). On the West face, Demec points were glued directly onto the longitudinal and horizontal reinforcement through recesses in the concrete cover allowing to measure the average strains of the reinforcing bars. On the East face, Demec points were glued onto the concrete surface as well as onto the two corner bars. The Demec points on the concrete formed a grid from which the strain distribution of the wall face could be

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**Fig. 4.** Test setup showing Test Unit WSH6 after failure.

**Fig. 5.** Instrumentation of the test units: Hard-wired instruments ((a) and (b)) and Demec points ((c) and (d)).
computed. Note that for Test Unit WSH1 a different instrumentation layout had been used, which involved more D emoc measurements but fewer hard-wired instruments. A detailed description of this instrumentation can be found in the document accompanying the data files [6].

The loading history corresponded to the standard protocol by Park [10]. The first step of the testing protocol was the application of the axial load, which was kept constant throughout the testing. Subsequently, the horizontal cyclic displacement history was applied to the wall head by the actuators and comprised two cycles at each ductility level (Fig. 6). The first two cycles were force-controlled while the following were displacement-controlled. In the first two cycles the wall was loaded until the actuator force equalled 3/4 of the nominal yield force \( F_y \) obtained from a moment–curvature analysis of the base section. The nominal yield displacement \( \Delta_y \) was then determined as 4/3 times the average peak displacement reached during these first cycles (this is the so-called 3/4-rule according to [11]). Starting from displacement ductility \( \mu_\Delta = 2.0 \) the amplitude of the displacement cycles was continuously increased by \( \Delta_y \) until failure occurred. At each ductility level the wall was subjected to two full cycles. A small loading velocity was chosen to keep dynamic effects to a minimum and to allow a continuous control of the instrumentation and the hydraulic system. For larger amplitude cycles the loading velocity was gradually increased as reported in Fig. 6.

The experiments showed that at 0.75 \( F_y \) the crack patterns of the six test units were not equally developed. For walls for which the axial load contributed significantly to the moment resistance (e.g. WSH5) the crack pattern was less developed than for walls with large flexural reinforcement ratios (e.g. WSH3) and therefore the estimates of the yield displacement of the walls differed considerably. The contribution of the axial load to the moment resistance is quantified by the parameter \( \varepsilon_{Ny} \), which is listed for all test units in Table 1. Once the tests were completed, the yield displacements were re-evaluated according to a second approach. In this second approach the displacement at first yield \( \Delta_y' \) was determined from the experimental results at the instant when in the test the first yield moment \( M_y' \) was reached at the base of the wall. \( M_y' \) was obtained from section analysis and corresponds to the moment for which a strain \( \varepsilon_c = R_{p02}/E_c \) at the corner reinforcing bars or a strain \( \varepsilon_c = 0.002 \) at the compressed concrete edge was reached, whichever occurred first [12]. The nominal yield displacement \( \Delta_y \) was then determined as \( \Delta_y = \Delta_y'/M_y/M_y' \) where \( M_y \) was also obtained from the section analysis. The nominal strength \( M_y \) corresponded to a strain of \( \varepsilon_c = 0.015 \) of the corner reinforcing bars or a strain of \( \varepsilon_c = 0.004 \) at the compressed concrete edge, whichever occurred first. It is believed that the latter is a more appropriate estimate of the yield displacement than the first [12]. The results listed in Table 4 illustrate the large variability associated with determining the yield displacement and therefore also with the displacement ductility capacity of RC walls.

In the following, drift values will be used instead of displacement ductilities for quantifying the deformation of the test units. The drift values are unambiguously defined as \( \delta = \Delta/L_0 \) and allow therefore a clear comparison between the test units.

3. Test observations

3.1. Hysteretic behaviour and failure mechanisms

The force–displacement hysteresis curves including the drift and displacement ductility values of the six test units are shown in Fig. 7. Photographs of the test units after failure showing the crack pattern and the extension of the cover spalling are depicted in Fig. 8. In the following, a brief description of the evolution of the damage and the final failure mechanism is given for each of the test units. Failure of a test unit was defined as a 20%–drop of the resistance at peak deformation compared to the maximum strength reached during the test [10]. The top displacements of the test units at failure are summarised in Table 4. Where applicable, the table also includes the top displacements corresponding to fracture of the longitudinal web reinforcement. However, fracture of the web reinforcing bars did not lead to a 20%-drop in resistance. Testing was therefore continued but the fracture of the web reinforcement constitutes an important damage state.

WSH1: Of all the test units, the longitudinal reinforcement of WSH1 had the poorest ductility properties. This applied in particular to the longitudinal web reinforcement, which showed almost no hardening (\( R_{p02} = 1.03 \)) and a very small strain capacity (\( A_{pt} = 2.34\% \)). As a consequence, the first bars of the web reinforcement ruptured at a drift as small as \( \delta = 0.68\% \) (load step LS26) because their ultimate strain was exceeded. At this stage the concrete cover at the wall base just started to show the first vertical cracks which always appear before the onset of the cover concrete spalling. Upon load reversal, several further web

### Table 4

<table>
<thead>
<tr>
<th>Test unit</th>
<th>( \Delta_y^a )</th>
<th>( \Delta_y^b )</th>
<th>( \Delta_y^c )</th>
<th>( \Delta_y )</th>
<th>( \Delta_{y, web-reinf.} )</th>
<th>( \Delta_{u} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSH1</td>
<td>10.5 0.23 8.4 0.18 11.0 0.24 31.0 0.68 47.5 1.04 4.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WSH2</td>
<td>10.5 0.23 7.8 0.17 10.5 0.23 48.1 1.05 63.0 1.38 6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WSH3</td>
<td>15.4 0.34 11.3 0.25 16.2 0.36 61.6 1.35 6.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WSH4</td>
<td>15.4 0.34 11.4 0.25 15.5 0.34 46.8 1.03 62.0 1.36 6.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WSH5</td>
<td>6.2 0.14 7.8 0.17 9.3 0.20 41.8 1.06 62.0 1.36 6.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WSH6</td>
<td>12.8 0.28 9.9 0.22 12.7 0.28 93.7 2.07 7.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **\( \Delta_y^a \)**: Yield displacement based on 3/4-rule. The loading history was based on this estimate for the yield displacement.
- **\( \Delta_y^b \)**: Displacement at first yield of the outer longitudinal reinforcement.
- **\( \Delta_y^c \)**: Nominal yield displacement based on strain limits according to [12].
- **\( \mu_\Delta = \Delta_y/\Delta_y \)**: A.Dazio et al. / Engineering Structures 31 (2009) 1556–1571
reinforcing bars ruptured (LS29). When unloading from LS36 to LS39 the wall was unintentionally pushed further in the positive direction and at $\delta = 1.04\%$ three of the innermost D10 mm bars of the North boundary element ruptured. The force dropped by 21% (56 kN) ending the test.

WSH2: The reinforcement layout of WSH2 was the same as for WSH1 but the ductility properties of the bars were better (see Table 2). This was directly reflected in the drift of $\delta = 1.16\%$ that caused the first fracture of web reinforcing bars (LS46, 6–8 bars fractured). This drift was larger than the ultimate drift of WSH1 but the bars of WSH2 still fractured at what is normally considered a quite low inelastic deformation since concrete cover spalling had not yet taken place. Upon load reversal a further 6–8 web bars fractured (LS49). By the end of the second cycle at the same deformation level (LS53) all the web reinforcing bars had ruptured within the continuous crack at about 175 mm above the base (see Fig. 8(b)). At this stage parts of the cover concrete fell off and revealed that the corner bars had buckled at approximately 150 mm above the base. At LS61 ($\delta = 1.39\%$) all six D10 mm bars of the boundary element in compression had buckled. Upon load reversal the first of these bars ruptured when a drift of $\delta = 1.11\%$ was reached. The test unit was pushed further and at $\delta = 1.80\%$ the second corner bar ruptured. Both bars had ruptured where they had previously buckled. As a consequence the capacity of the wall was reduced to 78% (279 kN) of its maximum capacity and the test was ended.

WSH3: For Central European standards Test Unit WSH3 represented a “model” ductile wall in terms of the reinforcement properties, the longitudinal reinforcement contents, and the detailing. The longitudinal reinforcement ratios, especially of the web, were significantly larger than the ratios of WSH1 and WSH2, and also the ductility properties of the longitudinal reinforcement were better. As a consequence no premature fracture of the longitudinal reinforcement occurred. Spalling initiated at $\delta = 1.02\%$ (LS26) but at this stage the longitudinal bars were not yet visible. As the displacement amplitudes were increased, concrete spalling continued and the longitudinal bars became first visible at a drift of $\delta = 1.70\%$ (LS51) showing already first signs of buckling. Despite the closely spaced confinement ties ($s/D_{nom} = 75/(12 = 6.25)$) the buckling of the bars became more severe in the following cycles. At $\delta = 1.79\%$, shortly before LS63 was reached, one of the D12 mm corner bars ruptured at about 140 mm above the base where during previous cycles the local curvature of the bar due to the buckling had been largest [13]. The test was ended at LS63 although the total drop in force capacity had been slightly less than 20%. The maximum drift reached by WSH3 was $\delta = 2.04\%$. Fig. 7(c) shows the very stable force–displacement behaviour of WSH3 with large hysteresis loops.

WSH4: The longitudinal reinforcement layout of WSH4 was the same as the one of WSH3. Also the ductility properties of the reinforcing bars of the two test units were very similar but unlike WSH3, WSH4 had no confining or stabilising reinforcement, i.e., it was not specifically designed for ductile behaviour. For WSH4 the onset of concrete spalling was observed at a drift of $\delta = 1.02\%$ (LS29). Since the longitudinal reinforcement was not stabilised by hoops or hooks, buckling effectively commenced at the same time as concrete spalling, which in return was further expedited. The first buckled bar became visible at LS31 ($\delta = 1.02\%$). In the following cycles spalling of the concrete continued, therefore reducing more and more the size of the concrete section near the edge of the wall, and the buckling of the boundary bars increased steadily. When loading to LS46 the test unit suddenly softened at a drift of $\delta = 1.04\%$. It regained stiffness but then the capacity started to drop: At the beginning the loss in force was moderate but at a drift of $\delta = 1.63\%$ the actuator force dropped suddenly by 63 kN. This drop in force had been caused by crushing of the unconfined concrete in the compression zone. The crushed concrete fell off reducing the effective wall length and therefore also its internal lever arm. The test unit reached a new equilibrium state with a lateral resistance of 331 kN which corresponded to 75% of the maximum attained actuator force ending the test. Up to failure the shape of the force–displacement hysteresis was very similar to that of the ductile Test Unit WSH3. Also the crack patterns of the two walls are very similar. However,
Fig. 8(d) shows that the damage to the compression zone of WSH4 was significantly more severe than the damage to the compression zone of WSH3.

WSH5: Test Unit WSH5 was designed for a similar moment capacity as WSH3 but since the axial load was about 2.14 times larger, its longitudinal reinforcement ratio had to be reduced. The spacing of the confining reinforcement of WSH5 was reduced to 50 mm because of the smaller diameter of the longitudinal reinforcing bars in the boundary regions \( \left( \frac{s}{D_{\text{nom}}} = \frac{50}{8} = 6.25 \right) \). The web reinforcing bars of WSH5 had poorer ductility properties compared to those of WSH3 while the ductility properties of the boundary reinforcement were similar despite the fact that D8 mm instead of D12 mm bars were employed for WSH5. The yield displacement \( \Delta_y = 6.2 \) mm used in the loading history had been determined according to the 3/4-rule which was later found not to be the most suitable method for determining yield displacements (Section 2.3). During the cycles with displacement ductilities \( \mu = 2-4 \) it was noticed that increasing the displacement amplitudes by just 6.2 mm would require an unrealistic number of cycles to reach failure. From LS44 onwards the increment of the displacement amplitudes was therefore doubled. Onset of spalling was observed for \( \delta = 0.55\% \) (LS36). Shortly before reaching LS46 (\( \delta = 0.82\% \)) problems with the load-follower occurred yielding relatively large oscillations in the axial load (the axial load history is included in the recorded data [6]). However, the first reinforcing bars only fractured at LS56 (\( \delta = 1.01\% \)) after these problems had been

Fig. 8. Crack patterns of the six test units after failure.
overcome. At this stage the first two to four web reinforcing bars fractured and the corner bars became visible showing already slight signs of buckling at a height of 110 mm above the base. When the wall was pulled in the negative direction (LS59) a further two to four web reinforcing bars fractured and also on this side the corner bars in compression became visible and buckled slightly. During the second cycle at $\delta = 1.01\%$ all remaining bars of the web reinforcement fractured along the continuous crack located about 200 mm above the base (Fig. 8(e)). In the following cycle (LS65 to 70) at a drift of $\delta = 1.35\%$ concrete spalling continued and also the other bars of the boundary elements became visible and buckled. When loading to LS71 one D8 mm bar fractured ending the test. Fig. 7(e) shows the pinching of the hysteresis curve due to the high axial force and the low longitudinal reinforcement content.

**WSH 6**: Test Unit WSH6 was subjected to the same axial load level as WSH5. However, WSH6 had the same longitudinal reinforcement layout as WSH3 and hence its moment capacity was the largest of all the six walls. The confining reinforcement was spaced at 50 mm ($s/D_{nom} = 50/12 = 4.17$) and due to the larger depth of the compression zone, the confined area of WSH6 also included the first row of the web reinforcing bars. Concrete cover spalling started at a drift as small as $\delta = 0.57\%$ (LS16) and at $\delta = 0.85\%$ (LS31 and LS33) the corner bars at both wall ends became visible but no signs of buckling could yet be detected. The onset of buckling did not occur until the cycles at $\delta = 1.70\%$ (LS59 and LS61). When loading to LS66 the force started to drop at a drift of $\delta = 1.87\%$. At $\delta = 2.11\%$ the actuator suddenly reached its end position and the loading direction had to be reversed. At a drift of $\delta = 2.11\%$ the state of the wall was very critical and it is believed that the wall would have failed had the loading continued a little further. When the wall was pulled in the negative direction the actuator force increased steadily until $\delta = 1.77\%$, then it started to decrease slowly and suddenly dropped to 120 kN at $\delta = 1.93\%$, which ended the test. The abrupt failure was caused by crushing of the compression zone due to fracture of a number of confining hoops.

### 3.2. Crack patterns

The amount and extension of flexural and flexure-shear cracking differed substantially from one test unit to the other (Fig. 8). To help understanding this difference, Table 5 compares theoretical cracking shear strengths $V_{th,c}$ and theoretical cracking moments $M_c$ evaluated for the geometry and the material properties of the test units to the maximum shear forces $V_{max}$ and the maximum bending moments $M_{max}$ at the base of the walls measured during the tests. The cracking moment $M_c$ indicating the onset of flexural cracking is calculated estimating the tensile strength of concrete according to EC2 [9] since the latter was not determined experimentally. $V_{th,c}$ is the shear resistance of members without shear reinforcement [9]. For members with shear reinforcement like the walls tested here, $V_{th,c}$ can be interpreted as the shear force for which the onset of flexure-shear cracking is expected. For all test units, $M_c$ corresponds to 25%–40% of $M_{max}$, hence flexural cracking was expected to take place. The height over which these cracks extended was the largest for WSH3 featuring the smallest $M_c/M_{max}$-ratio while the opposite holds for WSH5. The location of the primary flexural cracks was dictated by the shear reinforcement and for all test units except WSH4 and WSH5 secondary flexural cracks occurred in the boundary regions where the confinement ties were placed. In the case of WSH5 the longitudinal reinforcement content of the boundary region was not large enough to trigger new cracks while WSH4 did not feature confinement ties. It will be shown in Section 4.3 that the location of primary and secondary flexural cracks had a significant effect on the failure of Test Units WSH1, WSH2, and WSH5.

The maximum shear force $V_{max}$ measured during the tests on walls WSH3, WSH4, and WSH6 is significantly larger than $V_{th,c}$ and therefore extensive flexure-shear cracking took place. On the contrary, the flexure-shear cracking of Test Units WSH1, WSH2, and WSH5 was limited reducing the spread of plasticity and increasing local deformations near the base of the walls. These effects are discussed in a quantitative manner in the next section.

### 4. Discussion of the test results

The extensive instrumentation presented in Section 2.3 allows the estimation of local deformations of the test units. In the following, their effect on the global deformation behaviour of the test units as well as on the observed failure mechanisms is described.
4.1. Displacement components

The instrumentation of the walls allowed one to determine the contributions of the flexural and shear deformations to the total displacement. The flexural deformations were computed by double-integrating the curvatures computed from the LVDT chains along both wall edges assuming plane sections remaining plane and the shear deformations were evaluated from the diagonal measurements according to the method by Hiraishi [14]. The displacement components were evaluated for all peak displacements. Fig. 9(a) shows as an example the displacement components of Test Unit WSH6. The total flexural deformations $\Delta f$ are further broken down into the fix-end deformations $\Delta f_0$ stemming from the opening of the base crack right above the foundation and the actual flexural deformations $\Delta f_i$ of the wall above the base crack. The fix-end deformations $\Delta f_0$ were computed from the local deformations measured by the LVDTs “W-V-1-North” and “W-V-1-South”. The actual flexural deformations $\Delta f_i$ result from all other LVDTs. The summation of these three components yields total deformations that agree well with the directly measured top displacements (LVDT “W-H-1” in Fig. 5).

The ratios of the shear to the total flexural displacements for all the six test units are plotted in Fig. 9(b) showing that at the peaks of the cycles the ratio of shear to flexural displacements remains approximately constant throughout the loading history. The smallest ratio was observed for WSH5 ($\sim 0.05$) and the largest for WSH3 ($\sim 0.12$) while the remaining four test units yielded fairly similar ratios around 0.10.

4.2. Flexural deformations

All six test units failed in a flexural mode and for all six test units the flexural deformations were considerably larger than the shear deformations; for these reasons the flexural deformations are investigated in more detail in this section. The flexural deformations are examined regarding their distribution over the height of the wall with the objective of illustrating that for the same top displacements the local strain demands vary considerably between the six test units. Furthermore, it is shown that the flexural tensile force resultant is an important indicator for the spread of the inelastic deformations over the height of the wall and therefore on the local strain demand and the resulting failure mechanism.

Fig. 10 displays for different load steps the distribution of the average strains measured at the same height at the two wall ends. The average strains were obtained either from LVDT or from Demec measurements, like the strain profiles shown in Fig. 10. Strictly speaking, this is not correct if inclined flexure-shear cracks are present since in this case plane sections do not remain plane. Nevertheless, considering that curvatures are often used in the design and assessment of structural walls, the experimentally determined curvatures profiles are plotted in Fig. 11 and will be further discussed in Section 5. For Test Units WSH2 to WSH5 the curvatures were computed from the LVDT chains. Note that data obtained from the lowest LVDTs “W-V-1-North” and “W-V-1-South” is not included in the plots since due to the strain penetration into the foundation it is not possible to define a meaningful base length for these devices.

For Test Unit WSH1 the curvatures were computed from the Demec measurements on the concrete surface because WSH1 was not equipped with vertical chains of LVDTs. Cross checks have, however, shown that curvatures obtained from the LVDT chains, Demec measurements on the concrete surface and Demec measurements on the reinforcing bars agrees generally rather well. The largest differences between the three methods were observed at the base of the wall where due to the large deformation gradients, the slightly different locations of the gauges led to considerably different strain measurements. Fig. 11 shows that Test Units WSH3, WSH4, and WSH6 experienced larger curvatures higher up the wall compared to WSH1, WSH2, and WSH5. As a result, for the same top drift, the latter show larger curvatures near the wall base. This trend is better illustrated by Fig. 12 showing which parts of the wall contribute most to the flexural top displacements $\Delta f_i$. This is expressed by the flexural top displacement component $\Delta f_0$ at the height $h_i$.

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Fig. 10. Average strains along the four corner bars of the test units at different load steps for displacements to the South and to the North. To allow a cross-comparison between the test units the drift $\delta$ for all load steps is given. The Demec gauges 10–18 are located on the South-East (SE) corner bar (SW: 20–28, NE: 30–38, NW: 40–48).

were both obtained from moment–curvature analysis. In the moment–curvature analysis the confined and unconfined concrete were modelled according to the Mander-Model (see e.g. [1]) using measured average material properties for the unconfined concrete. For the reinforcement a multi-linear approximation of the measured stress–strain relationships was used. No failure criteria for both concrete and reinforcement were assumed and hence at high curvatures the bending moment remains almost constant with the only exception being Test Unit WSH4 where crushing of the compression zone is evident. Fig. 13(b) shows very clearly that the increase in the flexural tensile force resultant $T$ after yielding is significantly smaller for Test Units WSH1, WSH2, and WSH5 compared to the others. This is due to both their smaller longitudinal reinforcement contents $\rho_{tot}$ (see Table 1) and to the smaller hardening ratio of the reinforcement, in particular that of the web (see Table 2). Hence, for these walls the spreading of plasticity is expected to be limited as it was observed from the experimental results. These findings confirm that next to
Fig. 11. Curvature distribution over the height of the test units at different load steps. To allow a cross-comparison between the test units the drift $\delta$ for all load steps is given.

Fig. 12. Relative contribution $\Delta f_i / \Delta f$ of local flexural deformations $\Delta f_i$ occurring at the different elevations along the wall to the flexural deformation $\Delta f$ at the top of the wall. The relative contributions are evaluated for four selected drifts $\delta$, which are equal for all test units.

The extent of expected flexure-shear cracking the increase in the resultant force $T$ due to strain hardening should be considered when estimating the displacement demand of a structural wall and therefore it should find its way, for example, into equations for the plastic hinge length [16].

4.3. Premature fracture of the longitudinal web reinforcement

The behaviour of Test Units WSH1, WSH2, and WSH5 was characterised by a premature fracture of the longitudinal web reinforcement. This failure mechanism can be best explained considering Fig. 14, which shows a close-up of one wall corner of Test Unit WSH1 at the end of the test. The premature fracture of the web reinforcement of these three test units was caused by four main effects:

1. The reduced spread of plasticity discussed in the previous section resulted for the same top displacement in larger average vertical strains in the plastic zone when compared to the other test units.
2. The local crack pattern was clearly affected by the vertical reinforcement content and by the location of the horizontal reinforcement. In the web region additional confinement ties were not present, therefore the number of cracks was smaller than in the boundary region and their width was larger.
3. The small diameters of the longitudinal web reinforcement and the typical absence of concrete cover spalling ensured a better bond than that of the longitudinal reinforcement placed in the boundary region. As a result the strain concentration in the crack was larger for the web reinforcement.
Fig. 13. Results of the moment–curvature analysis for the test units. Ultimate curvatures (a) and (b) were computed assuming a strain limit of 0.6\(\varepsilon_y\) for all reinforcing bars [12], only for WSH4 the ultimate curvature was defined in correspondence of a concrete compressive strain of 0.004. 

4. The small hardening ratio of the web reinforcement increased the strain concentration at the crack even further and the small ultimate strain of these bars led finally to premature fracture. In Test Unit WSH1 all web reinforcing bars, which had poor ductility properties, fractured along an almost horizontal section located about 175 mm above the footing, which corresponds to the large crack shown in Figs. 8(a) and 14. Their contribution to the flexural strength of WSH1 was relatively small and their fracture did not immediately trigger the failure of the test unit. However, the resistance of this section was reduced and all following deformations concentrated within this crack. Hence, at this weakened section the strain demand on the boundary reinforcing bars increased disproportionally, which led to the premature fracture of some of the D10 mm bars although their ductility properties were significantly better than those of the web reinforcement. The same failure mechanism was observed for Test Units WSH2 and WSH5.

5. Plastic hinge analysis of the test units

Today, several sophisticated numerical tools for the analysis of RC structural walls under cyclic loading are available (e.g. [17–19]). Nevertheless, in seismic design and assessment the force–displacement relationship of a structural wall that is responding in the inelastic range is often derived by means of the plastic hinge method (see e.g. [1]). To design engineers the method appeals because: (i) it is simple, (ii) it is based on familiar design quantities like moments and curvatures, and (iii) it has already found its way into many seismic codes (e.g. into Eurocode 8, Part 3 [20]). In the most common formulations of the plastic hinge method the total displacement of the wall is computed as the sum of its yield displacement \(\Delta_y\) and a plastic displacement component \(\Delta_p\). However, in Section 4.1 it was noted that the ratios of shear to flexural displacements \(\Delta_s/\Delta_f\) of the test units remained approximately constant as the imposed displacement increased. For this reason a formulation of the plastic hinge method is chosen, which considers flexural and shear deformations separately where \(\Delta_s\) is obtained by multiplying \(\Delta_f\) with a constant factor. The total top displacement of a cantilever wall can therefore be written as [16]:

\[
\Delta = \Delta_f + \Delta_s = (\Delta_{yf} + \Delta_{pf}) \cdot \left(1 + \frac{\Delta_s}{\Delta_f} \right),
\]

When not available from experimental evidence like in the case here, the constant ratio \(\Delta_s/\Delta_f\) can be estimated for example according to the approaches presented in [16,21,12]. The plastic flexural displacement \(\Delta_{pf}\) is computed as a function of the plastic hinge length \(L_{ph}\) and the plastic curvature \(\phi_p\) at the wall base. Existing equations only vary regarding the assumed height of the...
centre of rotation; the simplest is the following, which assumes that the centre of rotation is located at the base of the wall:

\[
\Delta_y \phi_y = \phi_p - \phi_y = \phi_p - \phi_y L / L_y
\]

(2)

where \( \phi \) is the total base curvature, \( \phi_p \) the yield curvature at the base, and \( L_y \) the shear span of the wall. In seismic design the curvatures are obtained from moment–curvature analysis of the wall base section (in the following this is referred to as section analysis). Different formulations of the plastic hinge method propose different estimates of the plastic hinge length and different strain limits for concrete and reinforcement strains characterising chosen limit states, such as for example the onset of spalling of the concrete cover or the fracture of longitudinal reinforcing bars. Note that these strain limits and plastic hinge lengths are not necessarily directly related to quantities measured in the experiments; it is only important that when the strain limits in the form of the corresponding curvatures and the plastic hinge length are combined in Eq. (2), a good estimate of the wall displacement for the considered limit state is obtained. For this reason plastic hinge lengths and criteria for ultimate curvatures should not be interchanged between different formulations of the plastic hinge method.

Although not strictly necessary, it seems desirable to link the experimental results and the output of the section analysis on a more fundamental level than the top displacement. This can be done if both the plastic hinge length and the curvatures used in plastic hinge analysis are related to quantities that can be derived from the experimental results. In such an approach, the plastic hinge length should be related to the height of the plastic zone \( L_{p,y} \), i.e. the height of the wall over which inelastic deformations take place, and the curvatures from the section analysis should be connected to the local deformations of the wall. The following two sections outline a possible method to link curvatures from section analysis and plastic hinge length used in the plastic hinge analysis to experimentally-derived quantities.

5.1. Curvatures at limit states

An obvious method of relating the local deformations of the physical wall to the curvatures obtained from section analysis seems to be the direct comparison of strains. However, as it was shown in Fig. 10, the measured strains near the base are significantly affected by the inclined flexure-shear cracks of the wall. As a consequence of the fanned crack pattern the compressive strains in the concrete are larger and the tensile strains in the reinforcement smaller than strains obtained from (plane) section analysis. Hence, experimentally derived strains cannot be directly compared to strains obtained from section analysis.

A more suitable experimental measure of the local deformations at the wall base that can be linked to the results from section analysis is the base curvature. As outlined in the previous section, the base curvature cannot be easily measured directly since at the base of the wall the strain penetration into the foundation distorts the measured average strains. The base curvature shall therefore be estimated from the strain measurements above the wall base. Within the plastic zone of the test units the curvature profiles were approximately linear (Section 4, Fig. 11). Hence, to determine the base curvature a best-fit linear curvature profile over the height of the plastic zone \( L_{p,y} \) was determined and extrapolated to the base of the wall (Fig. 15(b)) [16,21–23]. The height of the plastic zone was estimated as the height above the foundation for which the curvature was equal to the yield curvature. From section analysis it was found that the latter was approximately \( \phi_y = 3 \text{ km}/\text{m} \) for all test units. This method of deriving the base curvature was applied to the curvature profiles computed from the LVDT measurements, which were shown in Fig. 11(b)–(f). Note that WSH1 was not equipped with LVDT chains along the wall height (see Section 2.3) and for the sake of simplicity it will not be included in the following discussion. Fig. 15(d) shows the base curvatures of Test Units WSH2 to WSH6 as a function of the top displacement. At each imposed ductility level the mean and the standard deviation of the base curvatures at the four peak displacements were computed. The plot shows that for the same top displacement the base curvatures of Test Units WSH2 and WSH5 are significantly larger than for the Test Units WSH3, WSH4, and WSH6. This is due to the strong concentration of deformations near the wall base for WSH2 and WSH5, which was already discussed in Section 4.2.

The damage observations made during testing (Section 3.1) allow the linking of different limit states to specific load steps (LS). The following five limit states are considered: (i) onset of spalling of the concrete cover, (ii) onset of buckling of the boundary reinforcing bars, (iii) fracture of the web reinforcing bars, (iv) fracture of the boundary reinforcing bars, and (v) concrete crushing in the compression zone. Naturally, not all limit states were attained by each test unit. Table 6 lists the load steps – expressed as top displacements – associated with the different limit states. From the results of the section analysis shown in Fig. 13 compressive strains \( \varepsilon_c \), tensile strains \( \varepsilon_t \), and total strain excursions \( (\varepsilon_c - \varepsilon_t) \) were extracted for the curvatures that are equal to the base curvatures computed from the LVDT measurements (Fig. 15(d)). Strains obtained in this manner can serve as estimates of strain limits appropriate to use in plastic hinge analysis in order to derive wall displacements corresponding to a given limit state. For comparison, the strains are also given for base curvatures that were obtained from Demec measurements on the reinforcing bars (Data Set (b) in Table 6) instead of LVDT measurements (Data Set (a) in Table 6).

The third set of data included in Table 6 (Data Set (c)) summarises average local strains obtained from Demec measurements. For all limit states except fracture of the web reinforcing bars the strains were obtained from Demec measurements on the corner bars. Compressive strains, which showed a strong gradient over the height, were computed from the Demec measurement between \( h = 60 \) and 210 while tensile strains, which did not vary that strongly near the base, were averaged between \( h = 60 \) and 360 mm. For fracture of the web reinforcing bars the Demec measurements on the concrete surface were used (\( h = 60 \) and 360 mm).

It is important to note that ultimate strains obtained from material tests cannot be employed directly as strain limits in plastic hinge analysis since the latter must – next to the mechanical material properties – also account for the approximations introduced by the plane-sections assumption and for differences between average local strains and maximum local strains. An example for the latter is the strain concentration in cracks observed for the web reinforcing bars of WSH2 and WSH5, which led to failure at average strains significantly smaller than the ultimate strain of the reinforcing steel (Section 4.3).

In the following the strains obtained from the section analysis (Data Sets (a) and (b) in Table 6) are discussed for each of the five limit states listed above and compared to the average local strains that are obtained directly from the Demec measurements (Data Set (c)).

- **Onset of spalling of the concrete cover:** At the onset of spalling of the concrete cover the compressive strains at the wall edge obtained from section analysis were for all walls around 0.3%. The strains obtained at curves from Demec measurements were slightly smaller than the strains at curves from LVDT measurements. However, for engineering purposes the difference is not significant and the results suggest that a suitable strain limit in section analysis for the onset of concrete cover spalling is 0.3%. The compressive strains obtained directly
Table 6

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Spalling</th>
<th>Buckling</th>
<th>Fracture web bars</th>
<th>Fracture boundary bars</th>
<th>Concrete crushing</th>
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<td>$\varepsilon_{c,\text{bar}}$ (10$^{-3}$)</td>
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<td>−4.3</td>
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<tr>
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<td>−3.0</td>
<td>78</td>
<td>−4.8</td>
<td>31.0</td>
</tr>
<tr>
<td>WSH4</td>
<td>47</td>
<td>−3.8</td>
<td>47</td>
<td>−3.5</td>
<td>20.1</td>
</tr>
<tr>
<td>WSH5</td>
<td>25</td>
<td>−2.8</td>
<td>49</td>
<td>−6.6</td>
<td>35.6</td>
</tr>
<tr>
<td>WSH6</td>
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<td>−3.4</td>
<td>78</td>
<td>−6.7</td>
<td>35.5</td>
</tr>
<tr>
<td>Mean</td>
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<td>−</td>
<td>−5.2</td>
<td>31.8</td>
</tr>
<tr>
<td>C.o.v.</td>
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<td>−</td>
<td>−0.28</td>
<td>0.22</td>
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</tbody>
</table>

(a) Strains from moment-curvature analysis at curvatures obtained from LVDT measurements

(b) Strains from moment-curvature analysis at curvatures obtained from Demec measurements

(c) Strains from Demec measurements

$^a$ Tensile and compressive strains at the location of the corner bars.

$^b$ Tensile strains at the location of the outermost web reinforcing bars.

from Demec measurements (DataSet (c)) are in average about 25% larger than the strains obtained from section analysis. This is consistent with the previously discussed finding that actual compressive strains near the wall base tend to be larger than compressive strains obtained from section analysis.

• Buckling of the boundary reinforcing bars: Restrepo [13] suggested that the total strain excursion $\varepsilon_t - \varepsilon_c$ is better suited for characterising the onset of buckling of reinforcing bars than the tensile strain $\varepsilon_t$ alone. In Test Unit WSH4 the bars of the boundary region were only stabilised by the shear reinforcement and as a consequence the buckling of the bars commenced upon spalling of the concrete cover, i.e., at much smaller strain excursions than for the other test units, which had narrowly-spaced confining reinforcement. A good measure for the effectiveness of the stabilisation of the longitudinal bars is the ratio of the spacing $s$ of the stabilising reinforcement to the bar diameter $D_{nom}$ of the longitudinal reinforcement. These ratios are summarised in the last column of Table 1 and varied between 4.17 (WSH6) and 7.50 (WSH2). The strains at curvatures obtained from Demec measurements show that the smaller the ratio $s/D_{nom}$, the larger the total strain excursion for which buckling commenced. For the strains at curvatures obtained from LVDT measurements the trend is less clear. However, both strains at curvatures from LVDT measurements and strains at curvatures from Demec measurements suggest that for plastic hinge analysis a strain limit of $\varepsilon_t - \varepsilon_c = 3.5\%-4.0\%$ is suitable for determining the limit state “onset of buckling” for capacity-designed walls if $s/D_{nom} \leq 6$. Note also that the total strain excursion that was obtained directly from the Demec measurements (DataSet (c)) were very similar to the strains obtained from the
section analysis at the curvatures that were obtained from Demec measurements. The difference results only from the facts that the linear curvature profile assumed for determining the base curvature is only an approximation of the true curvature profile and that the experimentally determined strains were measured at some distance to the wall base. However, as in the case of spalling of the concrete cover, the compressive strains resulting from section analysis were smaller than those measured in the test. The opposite holds true for the tensile strains.

- **Fracture of the web reinforcing bars:** Fracture of the web reinforcing bars was observed for WSH2 and WSH5 (WSH1 is not considered here). For both test units the tensile strains obtained from section analysis at the location of the outermost web reinforcing bar were approximately 2.0%–2.5%, which corresponds to 0.40–0.45$A_{gt}$. Strains obtained directly from Demec measurements (DataSet (c)) were even smaller and corresponded to about 0.35$A_{gt}$. The mechanism leading to the small ratio of average strain at fracture of the bar to the strain capacity $A_{gt}$ was discussed in Section 4.3. Since the mechanism depends strongly on the hardening ratio $R_m/R_{p02}$ more experimental results or a mechanical model are required before general recommendations regarding appropriate strain limits for plastic hinge analysis can be formulated. In the absence of either it is recommended to use a strain limit of 0.4$A_{gt}$ for plastic hinge analysis if the reinforcement content of the web and its hardening ratio are similar to those of WSH2 and WSH5.

- **Fracture of the boundary reinforcing bars:** This failure mode was observed for WSH2, WSH3, and WSH5 (WSH1 is not considered here). As for buckling, Restrepo [13] suggested that the fracture of bars is best related to the total strain excursions (note that at peak displacements the web reinforcing bars were never in compression since the compression zone did not extend beyond the boundary zone. For this reason the tensile strain only was considered when assessing the fracture of the web reinforcing bars). The total strain excursion obtained from section analysis at curvatures obtained from LVDT or Demec measurements were in average about 0.7$A_{gt}$, the scatter however, was relatively large. The strains that were obtained directly from the Demec measurements (Data Set (c)) varied largely between the four corner bars on which the measurements were taken. For this reason they were not included in Table 6. For plastic hinge analysis it is recommended to use a limit of $\varepsilon_s - \varepsilon_c = 0.5$–0.6$A_{gt}$, which accounts to some degree for the adverse effect of bar buckling at large inelastic deformations of the walls. A limit of $\varepsilon_s - \varepsilon_c = 0.5A_{gt}$ has also been proposed by Restrepo [24].

- **Concrete crushing:** Test Units WSH4 and WSH6 failed due to concrete crushing. The boundary zone of WSH4 was not confined and failure occurred therefore at a much smaller compressive strain than for WSH6, which had well-confined boundary elements. Strains associated with crushing of the concrete will, however, be strongly dependent on the concrete properties and the layout and the properties of the confining reinforcement. It is therefore not possible to deduce general strain limits suitable for plastic hinge analysis from the available experimental data. Note that for example for WSH6 the compressive strains obtained directly from Demec measurements at the onset of buckling were $-2.03\%$ while the compressive strains at the limit state “concrete crushing” obtained from section analysis was only $-0.79\%$. This underlines again the large difference between the compressive strains obtained from section analysis and average local strains measured on the wall.

### 5.2. Plastic hinge length

Based on the flexural deformations that were computed by double-integrating the curvatures obtained from LVDT measurements the plastic hinge length $L_{ph}$ can be computed according to Eq. (2). The resulting plastic hinge lengths are plotted in Fig. 15(e). In the previous section, strain limits for the plastic hinge analysis corresponding to given limit states were defined by extracting strains from section analysis corresponding to curvatures obtained from LVDT or Demec measurements. If these strain limits are used in conjunction with the plastic hinge lengths plotted in Fig. 15(e), good estimates of the top displacements are obviously obtained. However, in a design situation $L_{ph}$ needs to be estimated assuming a linear curvature profile over the height of the plastic zone $L_{pl}$ and a centre of rotation at the base. These assumptions have been proposed by Hines et al. [16] and yield the following equation for $L_{ph}$:

$$L_{ph} = 0.5 \cdot L_{pl} + L_{up} \quad (3)$$

where $L_{up}$ is the component of $L_{ph}$ that characterises the contribution of strain penetration to the top displacement. Hines et al. [16] also developed a mechanical model for estimating $L_{pl}$.

### 6. Summary and conclusions

This paper presents the results of six rectangular RC walls that were tested under quasi-static cyclic loading. The measurements taken during the experiments are made available online [6] and the tests can therefore serve the research community as a reference point.

The main aspects that were investigated by means of these tests concern the effects of the reinforcement content and of the ductility properties of the longitudinal reinforcing bars on the deformation behaviour of the walls and on their failure mechanisms as well as the relation of the experimental results to plastic hinge analysis. These aspects are particularly important for existing RC structures, which might have been designed according to capacity-design principles but with insufficient attention paid to the reinforcement contents and properties. The findings of this experimental study are summarised in the following:

- Walls with low longitudinal reinforcement content feature typically reduced flexure-shear cracking. In addition, the low longitudinal reinforcement content means that the flexural tensile force resultant $T$ of the cross-section of the wall is small and therefore also its increase due to hardening of the longitudinal reinforcement. This effect is further enhanced if the hardening ratio $R_m/R_{p02}$ of the longitudinal reinforcement is small. Both, the reduced flexure-shear cracking as well as the small increase of $T$ due to hardening of the longitudinal reinforcement lead to a concentration of plastic deformations near the base of the wall, reducing the displacement capacity of the wall.

- It is often assumed that the ductility properties of the longitudinal reinforcement in the boundary elements are more important than those of the longitudinal web reinforcement since the plane-sections—remaining-plane hypothesis suggests that the reinforcing bars in the boundary element are subjected to larger strains than the reinforcing bars of the web. However, it was shown that due to (i) the crack pattern, (ii) the poor ductility properties of the web reinforcing bars, and (iii) the good bond properties, the local strains in a crack might be larger for the web reinforcement than for the boundary reinforcement. For the six test units examined in this paper premature fracture of web reinforcing bars was observed for hardening ratios $R_m/R_{p02}$ $\leq 1.1$ and reinforcement contents...
of approximately 0.3% (WSH1, WSH2, WSH5) while the web reinforcing bars of Test Units WSH3, WSH4 and WSH6, which did not fracture prematurely, had hardening ratios larger than 1.20 and reinforcement contents of approximately 0.5%.

- Premature fracture of the web reinforcement typically occurs in one crack, which often crosses the entire wall. At the height of this crack, the wall is severely weakened and from the instant when the first web reinforcing bars fracture deformations tend to concentrate at that height. Hence, at this section the strain demand on the boundary reinforcing bars also increases disproportionately with the top displacements soon leading to the failure of the wall.

- Plastic hinge analysis is a tool that is often used in seismic analysis for determining the inelastic displacement capacity of structural members. The two key input variables are the plastic hinge length and strain limits associated with given limit states. Assuming a linear curvature profile over the height of the plastic zone, plastic hinge lengths and base curvatures associated with five different limit states were deduced from LVDT and Demec measurements taken during the experiments. From section analyses the compressive and tensile strains were evaluated at the experimentally determined base curvatures. In conjunction with plastic hinge lengths compatible with the assumed linear curvature profile, these strains can serve as indicators for appropriate strain limits in plastic hinge analysis of structural walls.

To conclude, the experiments showed the reduced deformation capacity of RC structural walls with low longitudinal reinforcement content. This effect was further increased if reinforcing bars with low ductility properties were used. When assessing the inelastic deformation behaviour of such walls by means of the plastic hinge method, the choice of the plastic hinge length should reflect the reduced spread of plasticity due to limited flexure-shear cracking and limited hardening. On the same matter, the choice of strain limits identifying specific limit states in the framework of plastic hinge analysis should consider the difference between maximum and average local strains and the differences between real strains that can be measured on the wall and strains derived from plane section analysis.

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References