Quasi-Static Cyclic Tests of Two U-Shaped Reinforced Concrete Walls
K. Beyer a, A. Dazio b, M. J. N. Priestley a

a European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), Pavia, Italy
b Institute of Structural Engineering, ETH Zurich, Zurich, Switzerland


To link to this Article DOI: 10.1080/13632460802003272

URL: http://dx.doi.org/10.1080/13632460802003272

PLEASE SCROLL DOWN FOR ARTICLE
Quasi-Static Cyclic Tests of Two U-Shaped Reinforced Concrete Walls

K. BEYER¹,², A. DAZIO², and M. J. N. PRIESTLEY¹

¹European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), Pavia, Italy
²Institute of Structural Engineering, ETH Zurich, Zurich, Switzerland

U-shaped or channel-shaped walls are frequently used as lateral strength providing members in reinforced concrete (RC) buildings since their form does not only provide strength and stiffness in any horizontal direction but is also well suited to accommodate elevator shafts or staircases. Despite this popularity, experimental results on the seismic behavior of U-shaped walls are scarce. For this reason a research program with the objective to provide additional experimental evidence for such walls under seismic loading was developed. It included quasi-static cyclic testing of two U-shaped walls at the structural engineering laboratories of the ETH Zurich. The walls were built at half-scale and designed for high ductility. The main difference between the two walls was their wall thickness. The project was chiefly focusing on the bending behavior in different directions and therefore the walls were subjected to a bi-directional loading regime. This article discusses the design of the test units, the test setup and the test predictions. Finally the main results are summarized in terms of failure mechanisms and force-displacement hystereses.

Keywords U-shaped Wall; Quasi-static Cyclic Test; Bi-directional Loading

1. Introduction

Over the last decades, the seismic behavior of reinforced concrete (RC) walls with rectangular cross sections has been the subject of extended research and several test series on such walls were conducted. In most of these, isolated walls were either subjected to quasi-static cyclic loading [e.g., Oesterle et al., 1976; 1979; Vallenas et al., 1979; Goodsir, 1985; Elnashai et al., 1990; Dazio et al., 1999], pseudo-dynamic loading [e.g., Thiele et al., 2000], or they were tested on a shaking table [e.g., Rothe, 1992; Yabana et al., 1996; Inoue et al., 1997; Sollogoub et al., 2000; Lestuzzi et al., 1999; Pinho, 2000; Reynouard and Fardis, 2001]. In addition, a small number of large-scale tests were performed in which rectangular walls were tested as components within a structural system [e.g., Panagiotou et al., 2006]. From the results of these experimental studies the key parameters controlling the behavior of rectangular walls under seismic excitation could be deduced and the results were adopted in code provisions providing detailed guidelines for design engineers.

Although U-shaped walls are very popular in practice, only very few experiments on U-shaped walls under seismic loading have been conducted in the past. To our knowledge, the largest test series carried out so far was a joint research project between the laboratories of Ispra and Saclay [Reynouard and Fardis, 2001]. Within the scope of that test series several specimens of a single U-shaped wall configuration were built and...
subjected to quasi-static cyclic and dynamic loading. The U-shaped wall was designed for medium ductility according to an older version of Eurocode 8 [CEN, 1994], taking, however, the proposed draft version from 2001 into account since the Eurocode 8 was under revision at the time of the design of the walls [Ile and Reynouard, 2005].

The test series which is described here aims at complementing the quasi-static cyclic tests from Ispra by testing U-shaped walls with different cross sections. Unlike the walls tested in Ispra, the test units of this project were not designed according to a particular code but their design for high ductility followed principles that were judged reasonable without being unnecessarily conservative with respect to the shear and sliding shear design. It was believed that such test units would give the best insight into the load transfer mechanisms of U-shaped walls and would help to reveal critical aspects in their behavior. In the framework of the project just two large scale test units could be tested, hence only a limited number of aspects could be investigated. It was decided to set the focus of the experiments on the behavior of U-shaped walls for displacements in different directions. Based on this focus the following tasks were identified:

- Assess the failure mechanism of U-shaped walls. In particular, observe whether the walls are susceptible to shear or sliding shear failure.
- Gain experimental evidence for the stiffness degradation as well as for the strength and deformation capacity of U-shaped walls when loaded in different directions.
- Determine the magnitude of the different deformation components (flexural, shear, and sliding component) of U-shaped walls for different directions of bending.
- Determine the torsional stiffness of the U-shaped walls (note, however, that due to space restrictions the torsional stiffness will not be discussed in this article).

These objectives were used as guidelines during the planning phase of the experiments. This article describes the analysis of the prototype (Sec. 2) on which the design of the two test units (Sec. 3) was based. The test setup and the loading history are described in Sec. 4. The test predictions are presented in Sec. 5 followed by the main test results, which are summarized in Sec. 6. The article closes with a discussion of the test results (Sec. 7) and conclusions from the experiments (Sec. 8). This article is focusing on the experimental part of the project. Apart from the test predictions, numerical results are not included here but results obtained from wide-column model analysis of the two U-shaped walls are presented in Beyer et al. [2008].

2. Investigation of the Prototype Behavior

In the test setup the two test units were cantilever walls fully fixed at the base and subjected to horizontal displacements imposed at the top by a set of three actuators. When planning the test units three main decisions regarding the test setup had to be made: (a) defining a cross section for U-shaped walls that is common in real design; (b) determining representative shear span ratios, i.e., the vertical location of the actuators; and (c) choosing an appropriate load stub, i.e., the connection of the actuators to the test unit, which best represents the restraints imposed on U-shaped walls in real building. To help make these decisions the seismic behavior of a fictitious 6-story reference building which included a U-shaped wall was investigated and the relevant results are briefly discussed in the following sections.

2.1. Reference Building

The lateral force resisting elements of the reference building comprise one U-shaped wall and a bi-directional moment resisting frame. The dimensions of the U-shaped wall are
typical for a shaft housing an 8-person elevator. An isometric and a plan view of the reference building are shown in Fig. 1. The assumed gravity load tributary area for the U-shaped wall is indicated in the plan view. The seismic hazard to the building was defined in terms of the elastic design spectrum of Type 1 for soil class B given in Eurocode 8 [CEN, 2003] with a design ground acceleration of 0.3 g and an importance factor of unity.

2.2. **Time-History Analysis**

In the quasi-static cyclic tests of the two test units the shear span, i.e., the ratio of moment to shear force, is constant during the experiment since the location of the actuator with respect to the base section of the wall is fixed. Under seismic loading, this ratio will commonly vary because real structures are multi-degree of freedom systems whose response during an earthquake is typically controlled by several modes. To decide on a realistic shear span ratio for the test units, a beam element model of the reference building was analyzed using the computer program Ruaumoko3D [Carr, 2004]. The input ground motions for the nonlinear time history analysis were three real, single-component records. These were chosen because of their corner period at about 2 s (Fig. 2a) which is the

![FIGURE 1](image_url) 6-story reference building: Isometric (a) and plan view (b), all dimensions in meters.

![FIGURE 2](image_url) Elastic design displacement spectrum and spectra of real scaled records used to analyze the reference building (a) ratios of base moment to base shear of the U-shaped wall plotted against the base moment for the two principal directions of the reference building (b and c).
corner period of the design spectrum according to Eurocode 8 [CEN, 2003]. The fundamental period of the building assuming cracked RC section properties is also close to 2 s, which implies that the expected displacement ductility demand was controlled by the constant displacement region. The records were scaled in accordance with the criteria given in Eurocode 8 [CEN, 2003]. They were then applied in direction of the principal horizontal axes of the building in two separate analyses. Figures 2b and c show the ratio of base moment to base shear of the U-shaped wall for the two principal horizontal directions plotted against the respective base moment. The term “principal directions” refers to the directions parallel to the web and parallel to the flanges of the U-shaped wall. The ratios were evaluated at 0.1 s intervals over the duration of the ground acceleration records. The comparison of Figs. 2b and c shows that the distribution of the shear span over time in the two principal directions was not the same: For the loading direction parallel to the flanges of the U-shaped wall (Fig. 2b), the framing action was more pronounced and consequently the effective height was in general smaller than for the loading direction parallel to the web (Fig. 2c). The shear spans which were finally used in the experiments are marked with black lines. These spans were chosen for several reasons: Firstly, only shear spans for which the full yield moment had been attained were considered. Secondly, from all the shear spans which fulfilled this first criterion the smallest was the preferred choice since the smaller the shear span the larger the shear force demand for the same moment capacity. The latter criterion allowed the testing of the walls under the largest shear force expected to occur when the walls undergo inelastic flexural deformations. This loading condition was deemed to be of particular importance for investigating the shear strength of the walls. Finally, practical aspects regarding the test setup had to be taken into consideration. For example, the height of the actuators in the two principal directions needed to be similar in order to limit the dimensions of the load stub to a reasonable size. A methodology for deciding on a representative shear span in quasi-static cyclic tests similar to the one described here was outlined by Dazio and Seible [2003].

2.3. Static Analysis

By means of static analyses the effect of different load stub designs on the behavior of the U-shaped wall was investigated. Two different designs of the load stub were considered: a solid top slab and a collar (Fig. 3a). If the load stub is constructed as a thick top slab warping at the top of the wall is completely restrained. If the load stub is designed as a collar warping is only partially restrained. Here the term collar refers to an increased wall thickness at the wall head which is required to fasten the actuators to the test unit. During the experiments the test units were mainly subjected to lateral displacements with the twisting of the wall head restrained; only at some instances a small twist was applied to determine the torsional stiffness of the test unit (see Sec. 4.3). To determine the most appropriate layout of the load stub, however, a torsional load case was chosen since the load stub was expected to have a larger influence on the torsional than on the flexural behavior of the test unit.

The effect of the load stub on the behavior of the U-shaped wall under torsional loading is illustrated by the following example: Fig. 3b shows the variation of twist over the height of a U-shaped wall with the dimensions of the prototype section when subjected to an arbitrary torsional moment of $T = 1,000 \text{kNm}$ at the effective wall height $h = 5.90 \text{m}$ (analytical closed-form solution for a cantilever wall according to Petersen, 1997). Linear elastic models were used since only small twists will be applied in the experiment. The results show that restraining warping significantly stiffens the wall. The restraint also influences the way the flanges carry the load since the top slab introduces an
in-plane force-couple at the top of each flange; the entity of the two counter-rotating force-couples is also called bi-moment [Heidebrecht and Stafford Smith, 1973]. If no top slab is provided, the top edge of the flanges is stress-free and the flanges carry the shear forces produced by the torsional moment mainly like cantilever walls in bending. In a real building floor slabs will provide partial restraint to warping and introduce bi-moments into the U-shaped wall at the story levels. Heidebrecht and Stafford Smith [1973] developed a simplified method to account for the warping restraint provided by floor slabs. To include this effect in the analytical solution the stiffness of one slab is smeared over the story height. Figure 3c shows the variation of twist over the height of the U-shaped wall of the reference building for four different cases. First, the U-shaped wall of the reference building is modeled to include the stiffening effect of the floor slabs. The system is analyzed for two types of loading for which closed-form solutions are available, i.e., an end moment ($T = 1,000 \text{kNm}$) and a constant, distributed moment ($mT = 1,000 \text{kNm}/20.4 \text{m} = 49.0 \text{kNm/m}^2$). The actual distribution of the torsional load on the U-shaped wall will vary during an earthquake; however, the two analyzed cases are likely to delimit the range of expected torsional load distributions. Secondly, the U-shaped wall up to its effective height is modelled. In the experiment, the wall can only be subjected to end moments. Hence, the system was analyzed for the two different end condition, i.e., restrained and unrestrained warping.

A comparison with the twist profiles of the models considering the full wall height shows that the wall with unrestrained top edge best matches the full wall height cases with regard to the shape of the twist profile and the torsional stiffness. For this reason the load stubs of the test units were designed as collars and not as top slabs.

### 3. Test Units

Based on the prototype elevator shaft which was described in the previous section two U-shaped test units were designed. The following sections describe their geometry and reinforcement layouts (Sec. 3.1) as well as the material properties (Sec. 3.2).
3.1. Geometry

The two U-shaped walls that were tested within the scope of this project were both half-scale models of the prototype elevator shaft. The principle parameter distinguishing the two units was the thickness of their walls. The wall thickness of Test Unit A was 0.15 m corresponding to 0.30 m at full-scale, while the wall thickness of Test Unit B was 0.10 m which corresponds to 0.20 m at full-scale. The wall thickness was chosen as parameter because it was expected to influence the overall behavior of U-shaped walls significantly. More specifically it affects—together with the concrete strength—the shear capacity of the wall, the compression zone depth, and hence the strain demand on the concrete. The former might become fairly deep for bending in the diagonal direction when only one flange end is in compression. In addition, the wall thickness controls possible out-of-plane shear, bending, and stability effects. These effects are difficult to model numerically and hence numerical parametric studies cannot substitute experimental results. Table 1 shows the cross sections and some characteristic values of Test Units A and B (hereafter called TUA and TUB) and the units tested in Ispra. In particular, the section of TUB is considerably less compact than the section of the units tested in Ispra and it is believed that the new experimental results will nicely complement the test series from Ispra.

### Table 1: U-shaped walls: Comparison of cross sections of TUA, TUB, and the units tested in Ispra

<table>
<thead>
<tr>
<th></th>
<th>TUA</th>
<th>TUB</th>
<th>Ispra</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scale</td>
<td>1:2</td>
<td>1:2</td>
<td>1:1</td>
</tr>
<tr>
<td>Shear span M/V</td>
<td>2.95m¹ / 3.35m²</td>
<td>2.95m¹ / 3.35m²</td>
<td>3.90m¹,²</td>
</tr>
<tr>
<td>Shear span ratio h/lw</td>
<td>2.81¹ / 2.58²</td>
<td>2.81¹ / 2.58²</td>
<td>3.12¹ / 2.60²</td>
</tr>
<tr>
<td>Axial load³</td>
<td>780kN</td>
<td>780kN</td>
<td>2120kN</td>
</tr>
<tr>
<td>Axial load ratio³ v</td>
<td>0.02</td>
<td>0.04</td>
<td>0.10–0.12</td>
</tr>
<tr>
<td>Compactness ratios:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>l_web/lw</td>
<td>8.7</td>
<td>13.0</td>
<td>6.0</td>
</tr>
<tr>
<td>l_fl/lw</td>
<td>7.0</td>
<td>10.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Vertical reinforcement ratio:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \rho_{vt} )</td>
<td>0.71%</td>
<td>1.01%</td>
<td>0.56%</td>
</tr>
<tr>
<td>Horizontal reinforcement ratio:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web: ( \rho_h )</td>
<td>0.30%</td>
<td>0.45%</td>
<td>0.54%</td>
</tr>
<tr>
<td>Flanges: ( \rho_h )</td>
<td>0.30%</td>
<td>0.45%</td>
<td>0.32%</td>
</tr>
</tbody>
</table>

¹NS direction (parallel to the flanges).
²EW direction (parallel to the web).
³at the wall base.
Figures 4a and d show the reinforcement layout of TUA and TUB. The longitudinal reinforcement was kept the same over the entire height of the wall and continuous bars running from the foundation to the collar were used (no lap splices). Only the spacing of the confining reinforcement was increased starting at 1.70 m above the wall base. The total area of the vertical reinforcement was approximately the same for the two test units (TUA: $A_s = 3281 \text{ mm}^2$, TUB: $A_s = 3224 \text{ mm}^2$). The horizontal reinforcement was identical for the two walls. Both the corners and the flanges were well confined in order to increase the compressive strength and, in particular, the compressive strain capacity of the concrete. The hoops in the corner boundary zones also ensured the integrity of the U-shaped wall.

As a measure against sliding shear failure the walls were equipped with shear keys at the wall bases (Figs. 4b and e). The shear keys were unreinforced concrete studs which were formed by moulds in the foundation. The foundation was cast before the wall. Prior to casting of the wall the moulds were lined with grease to prevent the fresh concrete from sticking to the moulds. When the wall was cast these moulds filled with concrete and thus provided an additional interlock between the wall base and the foundation (see also Sec. 5.2).

The shear spans of the test units were 2.95 m in the direction parallel to the flanges and 3.35 m in the direction parallel to the web (Figs. 4c and f). During testing of the walls a constant axial load, which was derived from the tributary area shown in Fig. 1b, was applied. The axial load at the base of the walls (i.e., including self-weight of the test unit and weight of any installations mounted on the wall) was 780 kN. It was applied close to the centre of inertia of the gross concrete sections (see Table 1).

### 3.2. Material Properties

Since the test units were half-scale models of the prototype shaft the maximum aggregate size was also reduced from 32 to 16 mm in order to limit scale effects to a minimum. For the reinforcement, bars with 6 and 12 mm diameter were used. Smaller diameters were not employed since in most cases the strain capacity of such bars is very limited. The concrete cylinder strengths of TUA and TUB at the day of testing were 77.9 and 54.7 MPa, respectively. In particular, the strength of TUA turned out to be significantly higher than intended. The objective had been to aim for $f_c = 45$ MPa which would have been representative for a concrete C30/37 according to CEN [2003] that is several years old. The larger concrete strength increased the shear capacity of TUA and hence rendered it less susceptible to shear failure. On the other hand, the large concrete strength reduced the number of cracks outside the boundary elements since the tensile strength of the concrete was also larger than expected. As a consequence, the crack widths were larger than for normal strength concrete which in return reduced the shear strength due to aggregate interlock. Because of the high concrete strength cracks often propagated through aggregates which further reduced the shear capacity due to aggregate interlock. However, all in all, the shear strength of TUA probably benefited from the high concrete strength. The large crack widths also placed a high strain demand on the longitudinal D6 bars. As a consequence, some of the D6 bars fractured towards the end of the test without prior buckling (Sec. 6.1). The yield strength $f_y$, tensile strength $f_t$, and strain $e_{su}$ at $f_t$ of the reinforcing bars are given in Table 2; stress-strain curves obtained from monotonic tensile tests are shown in Fig. 5. All reinforcing bars fulfilled the requirements for “Class C” grade steel according to Eurocode 8 [CEN, 2003].
FIGURE 4 TUA and TUB: Cross section, shear keys, and elevation of TUA (a-c) and TUB (d-f).
4. Test Setup, Instrumentation, and Loading History

Test setup, instrumentation, and loading history were designed in accordance with the objectives of the project, which were outlined in the introductory paragraphs. The following sections give an overview of these three aspects of the test design.

4.1. Test Setup

An isometric view of the test setup for TUA and TUB is shown in Fig. 6; a photo of the test setup is shown in Fig. 7. To control the two translational degrees of freedom and the twist of the wall head, the walls were loaded with three actuators: The EW actuator which loaded the web and the NS actuators which loaded the flanges. The alignment of the test units with the cardinal points as well as the labelling of different web and flange regions is shown in Fig. 8a. The force on the web (EW actuator) was applied at \( h = 3.35 \) m while the forces on the flanges (NS actuators) were applied at \( h = 2.95 \) m (Figs. 4c and f). The axial load was applied by a tendon which was pretensioned by a hollow core jack on a transfer beam on top of the test unit. The hollow core jack was connected to a load follower that kept the axial load constant throughout the entire test.

4.2. Instrumentation

During the experiments, the behavior of the walls was documented by taking notes and photographs as well as a large number of measurements. In total, 120 hard-wired instruments were installed which measured local and global deformations, strains of selected transverse reinforcing bars, as well as the applied forces. In addition, Demec measurements (Whitmore gauge measurements) were taken on the inside faces of the web and flanges in order to measure the deformation pattern of the lower part of the wall. Finally, the width of selected cracks was measured manually at instances of peak displacements and zero loads.
The most important measurements were the actuator forces, the displacements of the wall head, the sliding displacements at the wall base, the elongation of the wall edges, and the shear displacements (Fig. 9). The elongation of the edges was measured by four chains of linear variable differential transformers (LVDTs). Each chain consisted of
8 devices; the base length of the devices varied between 50 mm at the base and 1,000 mm at the top. The lowest device measured the variation in length between the foundation and \( h = 50 \) mm, i.e., the measured displacement included the opening of the base joint due to strain penetration into the foundation. The shear displacements were measured by diagonal string pots mounted on the outer faces of the web and the two flanges. Each face was covered by three crosses (Fig. 9); the first cross covered the height interval between \( h = 50–850 \) mm, the second between \( h = 850–1650 \) mm, and the third between \( h = 1650–2650 \) mm. With this instrumentation it was possible to determine the shear deformations of the web, the West flange and the East flange independently.
4.3. Loading History

As outlined in the Introduction, the key objective was to compare the behavior of the U-shaped walls for different directions of loading. Within this study, five different directions of loading were distinguished and labeled with letters (Fig. 8b):

- parallel to the web (Positions A and B);
- parallel to the flanges, flange ends in compression (Position C);
- parallel to the flanges, web in compression (Position D);
- in diagonal direction, one flange end in compression (Positions E and H);
- in diagonal direction, one corner in compression (Positions F and G).

When loading in the NS or in the diagonal direction the displacement was controlled at \( h = 2.95 \) m, for loading in the EW direction the displacement was controlled at \( h = 3.35 \) m. The diagonal direction was defined as the geometric diagonal of the section, which joins one outer corner with the opposite outer edge of the flange end. Hence, for the diagonal movement, the ratio of the displacements parallel to the web and flanges corresponded to the ratio of the web to flange lengths, i.e., 1.3:1.05.

Both test units were subjected to loading in all five directions according to a loading scheme based on a pattern developed by Hines \textit{et al.} [2002] who proposed a history comprising a “sweep” and a diagonal loading direction at each level. The particularity of the “sweep” is that a yield displacement is explicitly defined for the diagonal direction. The yield displacement in the diagonal direction is smaller than the vectorial addition of the yield displacements in the two principal directions, hence the pattern seems to be more rounded than the rectangular clover leaf pattern. The clover leaf pattern leads to large displacements in the diagonal directions and tends to impose higher ductility demands in the diagonal directions than in the principal directions. In the tests described here, the pattern developed by Hines \textit{et al.} [2002] was preceded by a full-cycle parallel to the web and a full cycle parallel to the flanges. The complete loading history for one cycle is hence (see Fig. 8b):

- EW cycle: full cycle parallel to the web (O→A→B→O);
- NS cycle: full cycle parallel to the flanges (O→C→D→O);
- diagonal cycle: full cycle in diagonal direction (O→E→F→O);
- “sweep” (O→A→G→D→C→H→B→O).

During these cycles the twist of the wall head was restrained. Only at Positions O, A, B, C, and D during the cycles at ductility levels 1.0 and 4.0 small twists were applied in order to determine the torsional stiffness of the walls. Altogether, the load pattern corresponds to three cycles parallel to the web and three cycles parallel to the flanges if the diagonal load cycles and the “sweep” are projected onto the principal directions. In terms of the number of plastic excursions (NPE) and the sum of normalized plastic deformation ranges (SNPDR) [Applied Technology Council, 1992; Krawinkler, 1996], the loading regime is hence more severe than the loading regime of the bi-directional test at Ispra where the wall was loaded with a full clover leaf pattern which corresponds to two cycles in the two principal directions at each ductility level [Pégon \textit{et al.}, 2000b]. The total loading history which was applied to TUA and TUB is the repetition of the load pattern in Fig. 8b at different ductility levels. The first four levels were within the elastic range of the wall. The amplitudes of these cycles were force-controlled with limits of 25, 50, 75, and 100% of the predicted lateral forces at first yield. During these cycles the sweep was replaced by the second diagonal (O→H→G→O). After the completion of the force-controlled cycles the nominal yield displacements in the five directions were
5. Analytical Consideration and Test Predictions

It was outlined in the Sec. 1 that the objective of this research project was not to validate any existing code design procedures but to contribute to the understanding of the behavior of U-shaped walls in general. Hence, no code was strictly followed during the design of the test units but several codes and research articles were consulted. From these approaches the authors either selected design equations which were believed most appropriate and not over-conservative or developed new approaches where existing models left open questions. In the following, the adopted design procedures for flexure and shear is briefly outlined.

5.1. Flexural Behavior

Plastic hinge analyses of the walls were carried out in order to predict the force and displacement capacities for the five different directions of loading (Sec. 4.3). Not all section-analysis programs were capable of analyzing U-shaped wall sections. In many programs, only I- and T-shaped standard sections are available which can be used to compute the moment-curvature relationship of U-shaped walls for bending parallel to the web and the flanges. For bending in the diagonal directions, however, it is necessary to model the U-shaped section explicitly and analyze it in directions different than the principal axes. For the analyses presented in this paper the program OpenSees [Mazzoni et al., 2006] was used. It allows the user to analyze any user-defined fiber section under any direction of loading. The constitutive relationships of the reinforcing bars followed the model by Giuffrè-Menegetto-Pinto; the stress-strain behavior of concrete was modeled using the constitutive equation by Popovics [Mazzoni et al., 2006]. Figures 10a and b show the moment-curvature relationships for TUA and TUB, respectively. Before the experiments were carried out only first estimates of the strain capacity of the reinforcing bars were available. From these preliminary tests the strain capacity was estimated as 10%. The curvature was therefore limited to a curvature associated with $0.6 \times 10\% = 6\%$ maximum steel strain [Priestley et al., 2007]. The reduction of the ultimate steel strain capacity by 40% accounts mostly for the effect of compressive strains.
and the reduction of the tensile strain capacity due to buckling. The plastic hinge length was estimated as the maximum of the two equations provided in Paulay and Priestley [1992]:

\[ L_{ph} = 0.08h + 0.022d_f f_y, \]  
\[ L_{ph} = 0.2l_w + 0.044h, \]

where \( h \) is the effective wall height, \( d_f \) the diameter of the main longitudinal reinforcement, \( f_y \) the yield strength in MPa, and \( l_w \) the wall length parallel to the loading direction. For the diagonal directions only the first equation was employed since the definition of a representative wall length for the diagonal direction is not evident; the effective height for bending in the diagonal direction was estimated as the average of the heights in the two principal directions. Figures 10c and d show the force-displacement predictions for TUA and TUB, respectively. The pushover curves include nominal shear displacements of 15% of the flexural displacements for all directions of bending.

5.2. Shear Behavior

Unlike the moment capacity, which in most cases can be computed fairly accurately if the material properties of steel and concrete are known, estimates for the shear capacity of walls—even with a simple rectangular shape—vary largely between different code provisions or shear models published in the literature. For U-shaped walls additional considerations regarding the distribution of shear forces among the different wall sections.
are necessary which introduces additional uncertainties. For the design of the test units
the shear force parallel to the web was entirely assigned to the web. The shear force
parallel to the flanges was distributed equally between the two flanges if the wall was
loaded parallel to the axis of symmetry. For diagonal loading the shear force was
assigned entirely to the flange in compression as suggested by Reynouard and Fardis
[2001]. The web and flanges of the U-shaped walls were then designed as independent
rectangular wall sections according to the modified UCSD shear model [Kowalsky and
Priestley, 2000]. The approach was complemented by a check on the capacity of the
compression strut according to the Swiss code for concrete structures [SIA, 2004]. On
the basis of these equations the shear reinforcement in web and flanges was determined
as two layers of D6 reinforcement at every 125 mm (see Figs. 4a and d).

For typical floor layouts the gravity-load area of a U-shaped wall is often of similar
size to the tributary area of a rectangular wall (Fig. 1b). Due to their large cross-sectional
area, when compared to rectangular walls, the axial load ratio $v = P/A_g f_c'$ of U-shaped
walls tends to be quite small. Walls with low axial load ratios are particularly prone to
sliding shear failure. Sliding interfaces are typically cold joints where the bond between
the two concrete sections is limited, e.g., at the interface between the foundation and the
wall. Since sliding was considered to be a potential failure mode care was taken that the
interface between wall and foundation was representative of such interfaces in real
structures. In order to achieve this, the wall and foundation were cast in two stages:
First, the foundation was cast upright. The surface of the foundation forming the interface
with the wall was roughened when the concrete began to stiffen. After a couple of days
the wall and the foundation were laid horizontally and wall and collar were cast as one
entity. Casting the wall upright as well was impossible due to height restrictions in the
concrete factory. According to the Eurocode 8 [CEN, 2003] both TUA and TUB were
supposed to fail in sliding shear. The authors believe that describing the sliding shear
failure as a friction-phenomenon at maximum lateral load (as the Eurocode 8 does)
egnores experimental evidence which has often shown that sliding failure of a wall
subjected to displacement cycles commonly occurs after load reversal prior to crack
closure in the compression zone—an observation already made by Paulay and Priestley
[1992]. For the sliding shear design of Test Units A and B a novel approach was
developed and when applied to TUA and TUB showed that both walls were susceptible
to sliding shear failure. As a consequence it was attempted to prevent sliding along the
interface between wall and foundation by introducing shear keys (see Figs. 4b and e, Sec. 3.1).

6. Test Results

The test results of TUA and TUB are presented in terms of the failure mechanisms
(Sec. 6.1), the force-displacement hystereses (Sec. 6.2) and the displacement components
(Sec. 6.3). Due to space limitations it is not possible to present all data that were collected
during testing. Therefore, the focus will be set on describing the global behavior of the
test units. Figure 11 shows the imposed displacement pattern as seen from the top. For
both TUA and TUB the actually imposed displacement pattern agrees reasonably well
with the target displacement pattern, which was shown in Fig. 8b.

6.1. Failure Mechanisms

6.1.1. TUA. Test Unit A failed due to fracture of the longitudinal reinforcing bars.
Buckling of the longitudinal bars was observed for the first time during the cycle in the
diagonal direction at $\mu_\Delta = 6.0$ when at Position E the D12 bar in the outer corner of the West flange buckled. The first bar fractures occurred during the EW cycle at $\mu_\Delta = 8.0$ when loading to Position B: Two D6 bars in the West flange ruptured; these bars had previously buckled since their cover concrete had been lost. The D12 bar that ruptured first was the bar that had first buckled; it failed when loading from C→D at $\mu_\Delta = 8.0$. The subsequent cycle in diagonal direction at $\mu_\Delta = 8.0$ was the final cycle: When loading to Position E (flange end in compression) two further D6 bars in the web that had buckled during preceding cycles ruptured. The two D12 bars at the West flange end, which had not yet ruptured, kinked markedly towards the East. Upon load reversal (E→F) these two bars ruptured. In addition, one further D12 bar and all remaining D6 bars in the West flange ruptured, eventually causing the failure of TUA. Figure 12 shows the fractured D12 bars at the West flange end and the state of the lower third of the wall at the point of failure.

Failure of concrete in compression was not observed for TUA. Onset of concrete spalling was observed at $\mu_\Delta = 3.0$ during the sweeping motion. Spalling at this stage was, however, still very limited and only became significant from the diagonal cycle at $\mu_\Delta = 4.0$ onwards. Until the end of the test the concrete in all four confined boundary elements was in fairly good condition whereas the unconfined concrete of the flanges and the web

**FIGURE 11** Imposed displacement pattern as seen from the top for TUA (a) and TUB (b).

**FIGURE 12** TUA: West flange end showing the ruptured D12 bars (a) and lower part of West flange and web (b) at Position F during the diagonal cycle at $\mu_\Delta = 8.0$ (point of failure).
seemed to “decompose” during the cycles of $\mu_\Delta = 8.0$, i.e., it did not form a solid unit anymore but was divided by cracks into pieces which could move relative to one another, especially when the relevant portion of the test unit was in tension. Due to the bi-directional loading history a complex crack pattern had formed in the web and flanges (Fig. 13). While

FIGURE 13  TUA: Crack pattern towards the end of the test (photos taken at Positions A and B during the EW cycle at $\mu_\Delta = 6.0$): South face (a), West face (b), North face (c), and East face (d).
the crack pattern of the web looked fairly similar to the typical crack pattern of a rectangular wall under cyclic loading, the flanges showed cracks of very different angles for the different directions of loading. The steepest cracks occurred at Positions E and H, respectively, within the flange which was in compression.

Throughout the test sliding shear displacements at the wall base were fairly small. At the peak displacement of the EW, NS, and diagonal cycles with $\mu_\Delta = 6.0$ the maximum sliding displacements of the web, the West and the East flange were 2.5, 1.2, and 1.6 mm, respectively, which corresponded to 4.4, 3.0, and 2.0% of the total top displacements at these instances. For cycles with smaller ductilities the sliding displacements were much smaller. As described before, during the cycles of $\mu_\Delta = 8.0$ the unconfined concrete near the wall base degraded considerably. Due to the relative movement of the pieces the sliding displacement measurements became less reliable and the intensive cracking caused some of the instruments measuring the sliding displacements to fall off. At the end of the test, when the base crack was about one centimeter wide in some parts, one could see that at least one of the shear keys in the West flange had sheared off. The condition of the other shear keys could not be inspected.

6.1.2. TUB. TUB failed due to crushing of compression diagonals in the unconfined part of the web when loading from H→B during the sweep at $\mu_\Delta = 6.0$. For TUB concrete spalling already initiated during the cycles of $\mu_\Delta = 2.0$ but longitudinal reinforcement did not become visible until the test unit was subjected to the NS cycle at $\mu_\Delta = 4.0$. As a result of the bi-axial loading history, spalling of the concrete spread from the boundary elements towards the unconfined regions. In the unconfined regions the loss of section width due to spalling was significantly larger than in the confined regions (Fig. 14). The reduction of the wall width eventually led to crushing failure of the web. In the region where the compression struts finally failed the wall width had in some parts been reduced to as little as $\sim$3 cm. After failure of the compression struts there were even a number of holes in the web that pierced through the entire web thickness. The crack pattern of TUB is shown in Fig. 15. In comparison to TUA the spacing of the cracks was smaller and the upper part of the wall had cracked more severely. In addition, the steep cracks forming in the West and East flanges at Positions E and H, respectively, were even slightly steeper than for TUA ($\sim$25° compared to $\sim$30°).

Unlike typical web-crushing failures, the failure of the wall was not very catastrophic since part of the lateral load could be transferred to the well-confined boundary elements at the corners of the test unit. These then acted as short columns while the web part above the failed compression strut acted like a kind of a “beam.” Figure 14 shows

![Figure 14](image-url)
the South and North face of the web at the point of failure. In particular, at the East edge a pronounced drift of the short column is clearly recognizable. The concrete of the confined corner elements was still in very good condition. After crushing of the compression diagonals the system did, however, become softer and the lateral load carrying capacity dropped by more than 20% with respect to the capacity attained at Position B at previous ductility levels. It was therefore decided to stop the test and to unload the wall.

**FIGURE 15** TUB: Crack pattern towards the end of the test (photos taken at Position B during the EW cycle at $\mu_\Delta = 6.0$): South face (a), West face (b), North face (c), and East face (d).
None of the longitudinal reinforcing bars of TUB had fractured when the wall failed due to crushing of the compression diagonal in the web. Buckling of the bars had, however, occurred and was first observed during the diagonal cycle (O\(\rightarrow\)E) at \(\mu_\Delta = 6.0\) for two D12 bars at the West flange end.

Sliding displacements at the wall base of TUB were even smaller than for TUA. The maximum sliding displacements measured just before beginning with the sweeping motion at \(\mu_\Delta = 6.0\) were 1.0, 1.7, and 4.0 mm for the web, the West and the East flange, respectively. These sliding displacements corresponds to 1.2, 2.2, and 4.4% of the total top displacements.

6.2. Hysteretic Behavior

The internal force distribution within U-shaped walls under bi-directional loading is very complex and requires some consideration during design. To give an idea about how the forces varied during the cycles at a constant ductility level, the actuator forces during ductility level 4.0 are presented in Fig. 16. In the figures the force-displacement data points which belong to peak displacements are marked with letters corresponding to the respective positions (Fig. 8b). The forces are considered as individual actuator forces. In addition, the moments and displacements in the EW and NS direction are combined in SRSS quantities. By definition, the SRSS moment and displacement are always positive quantities. For plotting hysteresis loops the SRSS moment and displacement are multiplied by the sign of the displacement in the NS direction. The SRSS moment and displacement are hence defined as:

\[
M_{\text{SRSS}} = \sqrt{M_{\text{EW}}^2 + M_{\text{NS}}^2} \cdot \text{sign}(\Delta_{\text{NS}}),
\]

\[
\Delta_{\text{SRSS}} = \sqrt{\Delta_{\text{EW}}^2 + \Delta_{\text{NS}}^2} \cdot \text{sign}(\Delta_{\text{NS}}).
\]

The following discussion focuses on the hysteretic history for the different directions of loading. Emphasis will be put on the cycles in the EW, NS, and diagonal direction; the sweep cycles are not included. For the three directions the hysteretic behavior is presented in separate graphs, i.e., each graph contains only cycles in one direction. The predicted force-displacement envelopes (Sec. 5.1) are also included in the plots. For TUA and TUB the force-displacement hystereses are shown in Figs. 17 and 18, respectively. For the diagonal cycles, four different hysteresis plots are presented: In the first plot (plot e) the hysteresis in the EW and NS direction are combined by using the SRSS moment and the SRSS displacement, which were defined in Eq. (6.1); in the three subsequent plots (plots f-h), the hystereses of the EW and NS actuators are plotted separately. In the following, the most important features of the force-displacement hystereses will be discussed. Unlike the failure mechanisms the main characteristics of the hysteresis curves of TUA and TUB are not very different and therefore only general remarks that apply to both walls will be made.

6.2.1. EW Cycles. Figures 17a (TUA) and 18a (TUB) show the force-displacement curve of the EW actuator for the EW cycles only. Although the force-displacement history is assembled using exclusively the EW cycles from the complex loading history, the hysteresis loops look similar to those of a symmetric rectangular or barbed wall.
that has been subjected to purely uni-axial cyclic loading. When loading in the EW direction the NS actuators were used to restrain the wall head from twisting. The forces applied by the NS actuators during the EW cycles are shown in Figs. 17b and 18b. The force couple was necessary since the shear center of the U-shaped wall laid North of the

Figure 16 TUA and TUB: SRSS moment-displacement hysteresis and force-displacement hystereses of the three actuators for the cycles at $\mu_A = 4.0$ for TUA (a-d) and TUB (e–h).
section and hence the applied EW force did not pass through the shear center. With increasing ductility demand on the wall, the magnitude of the required force couple reduced since the shear center moved closer to the center of the web [Pégon et al., 2000a].

FIGURE 17 TUA: Force-displacement hystereses for the cycles in the EW direction (a+b), in the NS direction (c+d) and in the diagonal direction (e-h).
6.2.2. NS Cycles. When a U-shaped wall is loaded parallel to its flanges (Positions C and D) the force distribution is simple because the wall is loaded parallel to its axis of symmetry: The lateral force in NS direction is split in equal parts between the two flanges while the force applied to the web diminishes. In reality, different states of cracking of the two
flanges and different stress states attained during the preceding cycles led to slightly different flange forces. For example, at Position C the West flange was typically softer and hence attracted less force. At Position B, which is the load position preceding the NS cycle, the West flange was in tension. Although the wall was pushed back to the zero position before starting the load cycle in the NS direction, the cracks in the West flange were still wider than in the East flange which had been in compression. As a consequence, at the onset of the NS cycle, the stiffness of the West flange was smaller than the stiffness of the East flange and therefore the West flange attracted slightly less force than the East flange.

Figures 17c and 18c show the force-displacement curves of the two individual NS actuators as well as the total force in the NS direction. Although the moment capacities in the positive and negative NS direction were approximately the same, the hysteresis loops look quite different in the two directions: For positive displacements, when the web was in compression, the hysteresis loops are fat. For negative displacements, when the flange ends were in compression, the loops are thinner and look similar to those for loading in EW direction.

If the wall had only been loaded in NS direction, theoretically no EW force would have been necessary in order to maintain zero EW displacement. However, at the beginning of a cycle in the NS direction the force in the EW actuator was always negative. This force resulted from the previous cycle in the EW direction. Since the wall was pushed back to zero EW displacement before starting the NS cycle, the EW force at the beginning of an NS cycle corresponded to the EW force in the EW cycle at zero displacement. This force decayed rapidly during the NS cycle and was usually close to zero when Position C was reached (Figs. 17d and 18d).

6.2.3. Diagonal Cycles. The direction of loading that is the most complex in terms of the load transfer mechanism is the diagonal direction. The main reasons for this are mentioned in the following:

- The stiffness of the flanges is different; the flange in compression is stiffer than the flange in tension.
- The locations where shear forces can be effectively transferred to the foundation are limited since the base crack is open along almost its entire length.
- Out-of-plane bending is likely to be a relevant part of the load transfer mechanism.

In Figs. 17e–h and 18e–h force-displacement hystereses from the cycles in the diagonal direction are shown. Figures 17f and 18f show the hysteresis loops of the EW actuator. There is a clear difference between the hysteresis curves in the diagonal direction and those in the EW direction (Figs. 17a and 18a): The loops are more pinched and in particular at Position E the maximally attained force is considerably smaller than at Position A. The opposite holds true for the total force applied in the NS direction (Figs. 17g and 18g): At Position E the maximum total force in the NS direction is only slightly smaller than at Position C while at Position F the total NS force is considerably smaller than at Position D.

The individual forces acting on the flanges and the web during the diagonal cycles were a result of the forces required to push or pull the wall along the diagonal direction and a force couple that was applied by the NS actuators in order to restrain the wall head from twisting. At Position E (i.e., when the West flange end was in compression) the forces from these two sources added up to a large force in the West flange and a small force in the East flange. The individual actuator forces are shown in Figs. 17h and 18h. It is striking that when loading from O→E the maximum force in the East flange was not
attained at the peak displacement but shortly after the zero displacement. The force then dropped continuously and at the peak displacement it was much smaller than the force in the West flange which was in compression.

At Position F, when the East corner was in compression, the force couple required to restrain the wall from twisting reduced the force in the compression flange (East flange) and increased the force in the flange under tension (West flange); the forces were therefore more equally distributed between the two flanges than at Position E. For TUA the force applied to the East flange at Position F was, however, still considerably larger than the force applied to the West flange (Fig. 17h) while the two actuator forces applied to TUB at Position F were of similar magnitude (Fig. 18h). Although the magnitude of the actuator forces was similar, the loading history was very different: The force in the East flange increased towards the peak displacement while the force in the West flange reached its maximum at a top displacement corresponding to $\mu_\Delta \approx 2.0$. For larger ductilities the force in the West flange remained approximately constant. The variation of the force in the West flange when loading to Position F hence shows a similarity to the variation of the force in the East flange when loading to Position E although the drop in force was considerably larger in the latter case.

Figures 17e and 18e show the SRSS moment plotted against the SRSS displacement. The shape of the loops is somewhat peculiar. As outlined at the beginning of this section, the SRSS moment and the SRSS displacement are by definition positive quantities. To produce Figs. 17e and 18e, the moments and displacements were multiplied by the sign of the NS displacement (Eq. 6.1). The jumps in the unloading branches results from the fact that the SRSS moment is not zero when the NS displacement is zero.

6.2.4. Comparison of Predicted Force-Displacement Envelopes to Experimental Results. For the EW and NS cycles the plastic hinge analysis predicted the force capacities of TUA and TUB (Figs. 17a,c and 18a,c) quite well. For the diagonal directions the situation is very different (Figs. 17e and 18e): The moment capacity in particular at Position E is largely overestimated by the plastic hinge analysis. The difference at Position F is less significant but still larger than for any of the principal directions of loading.

The assessment of the predicted displacement capacities is difficult since the displacement capacity of the other directions could no longer be determined once the wall had already failed in one direction. The point of failure of TUA at Position F is anticipated quite well by the plastic hinge analysis; TUB failed during the sweeping motion, which is not included in Figs. 17 and 18. With exception of Position C the displacement capacities predicted by the plastic hinge analysis are all smaller than the maximum displacements attained during the experiments. Hence, the displacement capacity predictions obtained from the plastic hinge analysis are in general conservative estimates.

6.3. Displacement Components

The total deformation of a RC wall is the sum of different displacement components. Three different displacement components are typically distinguished: sliding displacements at the wall base, flexural displacements (which include base rotations due to strain penetration into the foundation), and shear displacements. While the sliding displacements were measured directly at the base of the wall (Sec. 4.2) by means of LVDTs, flexural and shear displacements had to be computed from the LVDT measurements.
along the wall edges and from the diagonal string pot measurements. The flexural displacements were computed by double-integration of the curvature profiles obtained from the LVDT measurements. While the approach for computing the flexural deformations is applied consistently in various experimental wall studies, a number of approaches exist for the computation of shear displacements. The approach adopted in this study was first published by Hiraishi [1984] and accounts for the effect of the curvature distribution on the diagonal string pot measurements.

Figures 19 and 20 show the relative contributions of the different displacement components for TUA and TUB, respectively. The cycles in the EW, NS, and diagonal direction are plotted in different diagrams. For the EW cycles only the deformation of the web was analyzed (plot a) since the deformation of the flanges is of minor importance, while for the NS cycles only the deformation of the flanges was considered (plot b). For the NS cycles the displacement components are average components of the two flanges. For the diagonal cycles the displacement components of the web and the two flanges were analyzed separately.

FIGURE 19 TUA: Displacement components for cycles in the EW (a), NS (b) and diagonal (c-e) direction.
In all cases the sliding displacements do not constitute a significant part to the total displacements (see also Sec. 6.2). The ratio of shear to total displacements is considerably larger for TUB than for TUA whose wall thickness was 1.5 times the wall thickness of TUB. The most unexpected observation concerning the shear displacements was that at Position E in the East flange, they pointed in the opposite direction to the flexural displacements of the wall (Figs. 19d and 20d). In Sec. 6.2 it was mentioned that the force in the East flange dropped when the wall was pushed to Position E. It is likely that this caused the inverse sign of the shear displacements with respect to the total displacements.

A further striking finding is the large contribution of the shear displacements to the total displacements, particularly at Position F for the West flange (Figs. 19e and 20e) but also at Position E for the web (Figs. 19c and 20c). In both situations the entire flange and the web, respectively, of the U-shaped wall were under tension and it is likely that large crack widths led to a small shear stiffness and therefore to large shear displacements.

FIGURE 20 TUB: Displacement components for cycles in the EW (a), NS (b), and diagonal (c-e) direction.
7. Discussion of the Test Results

The results of the experiments on the two U-shaped walls were discussed in terms of failure mechanism, force-displacement hystereses, and displacement components. The experiments have shown that the adopted design approach led to a ductile behavior of TUA and TUB which reached displacement ductilities of 8 and 6, respectively. TUA failed due to rupture of the longitudinal reinforcing bars after buckling—a failure mode that is also often observed for well-detailed and capacity-designed rectangular walls. TUB failed due to web crushing. This was not the targeted failure mechanism even though the well-confined corner elements prevented a sudden collapse. The concrete crushing failure in the web was caused by the reduction of the effective wall thickness due to spalling of the unconfined concrete outside of the boundary elements. During the design it had not been considered that bi-directional bending might favor the spalling of concrete in these regions to such an extent.

The bi-directional loading history was fairly complex involving cycles in three different directions (EW, NS, and diagonal) and a sweeping motion. For the design of U-shaped walls it is important to understand the force distribution between the different wall sections (web and flanges). It was shown that in particular the force distribution between the two flanges during the diagonal cycles was very complex and originated from two sources, i.e., from the forces required to impose the lateral displacements and from the forces required to restrain the wall head from twisting. Comparison of analytical and experimental results has also shown that the maximum attained moment for bending in the diagonal direction was—in particular at Position E when one flange end was in compression—significantly smaller than predicted by the plastic hinge analysis while the plastic hinge analysis led to good estimates of the moment capacity for cycles in the EW and NS direction. More advanced modeling techniques that are capable of considering the loading history will be required in order to explain and capture the discrepancy in moment capacity between the experimental results and the numerical analysis. A detailed discussion of the behavior of U-shaped walls for loading in the diagonal direction is included in Beyer et al. [2008].

A further finding from loading the U-shaped walls in different directions concerns the contribution of the shear displacements to the total displacements: Depending on the direction of loading the ratio of the shear to the total displacements varied greatly between the different wall sections, i.e., the web and the two flanges. It was also found that the contribution of the shear displacements was largest when a wall section was under net tension. The magnitude of these shear displacements was larger than what is encountered in typical rectangular walls and might demand that shear displacements are explicitly accounted for in the design and analysis of U-shaped walls. Contrary to the predictions by Eurocode 8 [CEN, 2003] the sliding displacements along the interface of the wall and the foundation were at all stages of the experiment relatively small and did not trigger a failure of the wall.

8. Conclusions and Outlook

The objective of this article was to present the results of quasi-static cyclic tests on two U-shaped RC walls. Although U-shaped walls are frequently used in practice, experimental evidence is very scarce and it is believed that the two tests presented here will contribute to the characterisation of the seismic behavior of U-shaped walls and help to develop appropriate design guidelines. This is necessary since most of the recent seismic codes are still focusing on rectangular walls and the application of the guidelines to non
rectangular walls is often not straightforward but burdens the designing engineer with additional critical decisions such as how to distribute the forces between the flanges. The tests have also shown that the most critical direction is the diagonal one: For this direction of loading the maximum attained moment was less than what plastic hinge analysis would predict, while at the same time the displacement capacity is smallest. Most engineers, however, do not consider the diagonal directions when designing U-shaped walls; a circumstance that is also caused by the fact that many section-analysis programs cannot handle the analysis of U-shaped walls in directions others than the principal ones. Therefore a key objective of further studies that are currently underway is the development of simple analytical tools capable of predicting the moment and displacement capacities of U-shaped walls loaded in the diagonal direction.

Acknowledgments

Funding for the tests and partial financial support of the first author was provided by the ETH Zurich. The tests were carried out in the structural engineering laboratories of the ETH Zurich where C. Gisler and T. Jaggi were instrumental in the assemblage of the test setup. M. Baumann designed and implemented the test control and provided support during the instrumentation and the actual testing of both units. M. Trüb reviewed the first draft of the manuscript. Both anonymous reviewers provided very helpful and constructive comments.

References


SIA [2004] Betonbaut (Reinforced concrete), Swiss Standard SN 505 262, Schweizerischer Ingenieur- und Architekten-Verein, Zürich, Switzerland.

