

From experimental strain and crack distributions to plastic hinge lengths of RC walls with SMA rebars

Ryan Hoult^{*}, João Pacheco de Almeida

Institute of Mechanics, Materials and Civil Engineering, UCLouvain, Louvain-la-Neuve, Belgium

ARTICLE INFO

Keywords:

SMA
Core
DIC
Seismic
Residual-based
Strain distribution
Plastic hinge
Crack widths

ABSTRACT

Modern-designed reinforced concrete walls can typically achieve the primary performance level of collapse prevention in a moderate-to-large earthquake event. However, the substantial amount of damage sustained by the wall can result in large residual displacements of a building, often requiring costly repairs or even demolition. Localised replacement of reinforcing steel for a pseudoelastic material, such as shape memory alloys, has been offered as a potential solution for reducing residual displacements of reinforced concrete walls. This paper examines two basic assumptions that can be employed for the simulation of reinforced concrete walls with shape memory alloy rebars using distributed and lumped plasticity line models, namely: plane-sections-remain-plane and plastic hinge length. To this purpose, the authors analyse the strain and crack distributions using the experimental data from a recent experimental campaign. Digital image correlation techniques were used to capture the displacement field of the surface of the walls. Experimental findings are presented, including crack distributions and salient crack widths, longitudinal strain profiles along the wall length and height, and curvature profiles. They are used to discuss the validity of the plane-section hypothesis and equivalent plastic hinge length expressions. A conventional wall reinforced with steel rebars is used for comparison purposes. The experimental results presented in this paper are useful for the ongoing international investigations on concrete structures detailed with shape memory alloys.

1. Introduction

Reinforced concrete (RC) walls form the backbone of millions of mid- and high-rise buildings around the world. In the event of strong ground motions from an earthquake, the RC walls provide the primary resistance against the generated lateral forces due to their large relative in-plane stiffness. In modern earthquake engineering, RC walls are designed with the intention of providing sufficient strength and ductility (i.e., large but controlled deformations) with the aim of avoiding collapse for a rare earthquake [67]. To provide the required strength, modern walls are designed with additional (“lumped”) reinforcing steel at the boundary ends of the wall. The steel bars in these “boundary regions” are expected to perform inelastically to provide the required deformation capacity and energy dissipation. However, the potentially high plastic demands of the steel reinforcement in these boundary regions results in permanent damage and is typically the source of the residual displacements in RC buildings (i.e., the permanent relative deformation of a structure with respect to its foundation). For example, 25% of buildings in Christchurch, New Zealand were no longer vertical

after the M_w 6.3 earthquake in 2011 [57], where 60% of RC buildings over three storeys were demolished because of extreme building tilts and being declared “uneconomic to repair” [32]. While the primary performance level of no collapse for modern-designed RC structures can typically be achieved, the level of damage to structures and the economic losses can be unexpectedly high [31]. In the case of Christchurch, losses of more than \$NZD 40 billion (i.e., 20% of the country’s GDP) were accumulated from the event [50]. The current design philosophy of collapse prevention (or life safe), while deemed satisfactory to most engineers, no longer appears to satisfy the request of building owners and the public at large, whom are overwhelmed by the extent of damage, cordoning off of the city, and disruption to services [31]. Thus, in addition to collapse prevention as a performance objective, more attention needs to be paid – even after rare earthquakes – to the post-earthquake reparability and serviceability of structures. Both can be controlled by the peak inelastic displacement demand during the dynamic structural response, but residual displacement is also an important criterion, and the one directly measurable after shaking. An abundance of evidence in the last couple of decades, largely from post-

^{*} Corresponding author.

E-mail address: ryan.hoult@uclouvain.be (R. Hoult).

<https://doi.org/10.1016/j.engstruct.2022.114731>

Received 21 March 2022; Received in revised form 26 June 2022; Accepted 22 July 2022

Available online 29 July 2022

0141-0296/© 2022 Elsevier Ltd. All rights reserved.

earthquake reconnaissance missions [32,43,54], has attracted an increasing number of analytical and experimental research focusing on reducing residual displacements. Design and assessment provisions considering residual displacement checks are being included in modern international codes, either explicitly or through indirect verifications, with recent studies indicating that a maximum permissible residual drift (i.e., ratio of the permanent building displacement to the building height) of 0.005 rad ($\approx 0.3^\circ$) could be used [52]. A recent investigation by the authors used an extensive number of experimental results from RC wall specimens to determine the relationship between the residual displacement and the in-plane drift [37]. These preliminary results reveal that conventional RC walls (detailed with steel) exceed, on average, the residual drift limit of 0.3° when subjected to in-plane drift demands greater than 1.5%. Therefore, to achieve more robust RC wall building performances and reduce residual drifts after large earthquake events, it is clear that engineers will not only have to design for larger earthquake demands but will also need to implement better structural technologies and materials [31].

Shape Memory Alloy (SMA) rebars have recently attracted some interest from researchers and structural engineers because of their ability to (i) recover displacements upon removal of stress, (ii) dissipate energy through hysteretic damping, and (iii) provide strength and displacement capacities comparable to conventional steel reinforcement [2]. It is for these reasons that SMA bars have the potential to be used as reinforcement in wall boundary ends, aiming to reduce residual displacements and ultimately decrease societal and economic impacts after seismic events. Although actual application of SMAs in civil engineering is still rare, there has recently been a considerable research effort in a wide range of seismic design applications, such as vibration control [73,74], base-isolation [14], and retrofitting [28,42]. There has also been some research using SMA bars to reinforce concrete structures, particularly in RC columns [7,10,64,65,66]. The authors also acknowledge the three wall specimens tested with SMA bars [2,21,45], which all experienced some experimental or laboratory limitations that essentially restricted the seismic performance of these specimens, including: (i) no axial load was applied to all three wall specimens, which has been shown to greatly reduce the residual displacements [24,37,49] and (ii) low shear spans (i.e., ratio of wall height to length) were used for all three specimens, representing low-rise buildings only. Regarding the latter of these issues, the construction and testing of considerably taller and more expensive units is not always possible [33].

An experimental program was undertaken at Université catholique de Louvain (UCLouvain) in the Institute of Mechanics, Materials and Civil Engineering (IMMC), where two large-scale (2:3) RC walls were subjected to quasi-static cyclic loading protocols. In one specimen, ribbed steel rebars were replaced by nickel titanium (NiTi) SMA bars over a limited height above the base and inside the wall boundary element regions. The employed test setup allowed imposing an axial load as well as a large shear span, forcing the walls to behave in flexure as representative of medium to high-rise buildings. The full experimental test setup, loading procedures, material properties, and initial test observations for the two wall specimens is described in Almeida et al. [5]. For sake of brevity, only a brief overview of the experimental test setup will be provided here. The primary purpose of this research paper is to examine two basic assumptions that can be employed for the simulation of reinforced concrete walls with shape memory alloy rebars, namely plane-sections-remain-plane and plastic hinge length. They are based on a critical analysis of new experimental findings from the tests, which were not examined or reported before, using the results from different instrumentation, including digital image correlation techniques that captured the displacement field of the surface of the wall specimens. Strain profiles are calculated from the displacement fields, which are also used to calculate curvature profiles and crack distribution with salient crack widths. These findings will help future validation and calibration of models that attempt to simulate the behaviour of these types of walls. They can also be used to modify or derive mechanical

models for the seismic assessment of RC walls, which will help to develop more robust guidelines in future building codes for the design of RC walls detailed with conventional steel and shape memory alloys.

2. Testing instrumentation and analysed experimental data

A summary of the tests by Almeida et al. [5] – focusing on the information required for this investigation – is given in the following sections, including the loading protocol and the instrumentation used. The original dataset from these wall tests, which is used for the subsequent analyses herein, is also publicly accessible through the UCLouvain *Dataverse* platform under the <https://doi.org/10.14428/DVN/2QBQVJ> [6].

Two wall units at 2:3 scale with identical geometry were tested. A RC wall detailed completely with conventional steel (herein denoted specimen RC) was designed in accordance to the Eurocode 8 [15], an identical RC wall was also tested but detailed with four SMA rebars at the base of each boundary element (herein denoted as specimen RC + SMA). Each wall unit was 2000 mm tall, 200 mm thick, and 1200 mm long, and included a top loading beam ($450 \times 400 \times 2400 \text{ mm}^3$) and a foundation block ($600 \times 400 \times 2800 \text{ mm}^3$). Fig. 1a illustrates the main geometry and the reinforcement layout of the two test specimens. For the RC + SMA specimen (Fig. 1a), SMA rebars made of nickel titanium (NiTi) alloy were used with a diameter $d_{bl} = 12.7 \text{ mm}$, which replaced the bottom 270 mm of longitudinal steel reinforcement in the wall. The NiTi rebars were selected such that their uniaxial strength after full martensitic transformation was smaller than the steel rebar yield strength, hence guaranteeing that the SMA cyclic pseudoelastic response could be well explored before yielding of the steel rebars. The application of these capacity design principles in the axial direction of the longitudinal rebars was completed by the simultaneous verification that the NiTi rebar ultimate strength was higher than the yield strength of the steel rebars. In this way, a spread of plasticity throughout the wall (above the SMA rebar) is also assured for larger inelastic displacement demands. Note that this phase of SMA response is accompanied by a regain in stiffness of the NiTi rebars in their martensitic phase, which contributes to limit the displacement demand in the wall during strong ground motions. Finally, the ultimate strength of the SMA rebars is also higher than the ultimate strength of the steel reinforcement, which avoids rebar failure in the SMA region. Mechanical couplers with an approximate length of 140 mm were used to connect the SMA and conventional steel rebars (Fig. 1c). More information regarding the specifics of the material and mechanical properties of the concrete, steel, and SMAs can be found in Almeida et al. [5].

The test setup was assembled at the technological platform *Laboratoire Essais Mécaniques, Structures et Génie Civil* (LEMSC) of the IMMC at UCLouvain. The setup is shown in Fig. 1d and consisted of two independent steel frames fixed to the laboratory strong floor, one supporting the vertical actuators and the other serving as a reaction frame for the horizontal actuator force. The vertical actuators apply an axial load of 350 kN to the specimens, corresponding to an axial load ratio (ALR) of approximately 3.0%, as well as applying a varied bending moment that corresponds to a constant shear span ($M/VL_w = H_e/L_w$) of 3.6. The out-of-plane displacement at the height of the RC beam was prevented by using sliding supports that were attached to lateral bracing frames (i.e., blue lateral frames in Fig. 1d). The horizontal actuator applied the reverse cyclic displacement history to the top RC beam of the specimens, which followed an exponentially increasing displacement envelope. Almeida et al. [5] and Steinmetz & Wyckmans [69] developed this *ad hoc* quasi-static loading protocol from the results of nonlinear dynamic analyses.

Throughout testing, the wall behaviour was monitored with conventional instrumentation, including linear variable differential transducers, potentiometers, inclinometers, and a laser to monitor global horizontal (in-plane) displacements of the loading beam. Nine Mitutoyo digital micrometre sensors (Fig. 2a) were also placed at the base of the

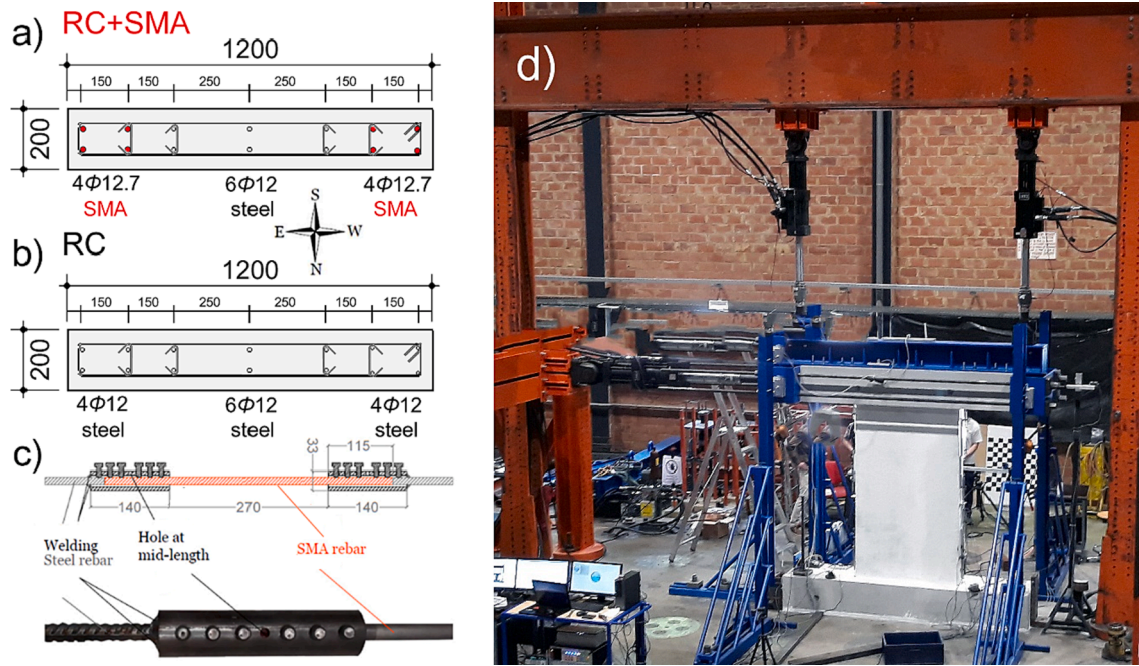


Fig. 1. Cross-section and longitudinal reinforcement layout of test units (a) RC + SMA and (b) RC, (c) SMA-steel rebar assembly and mechanical coupler photo, and (d) test setup assembly at the technological platform LEMSC for the test of the two walls.

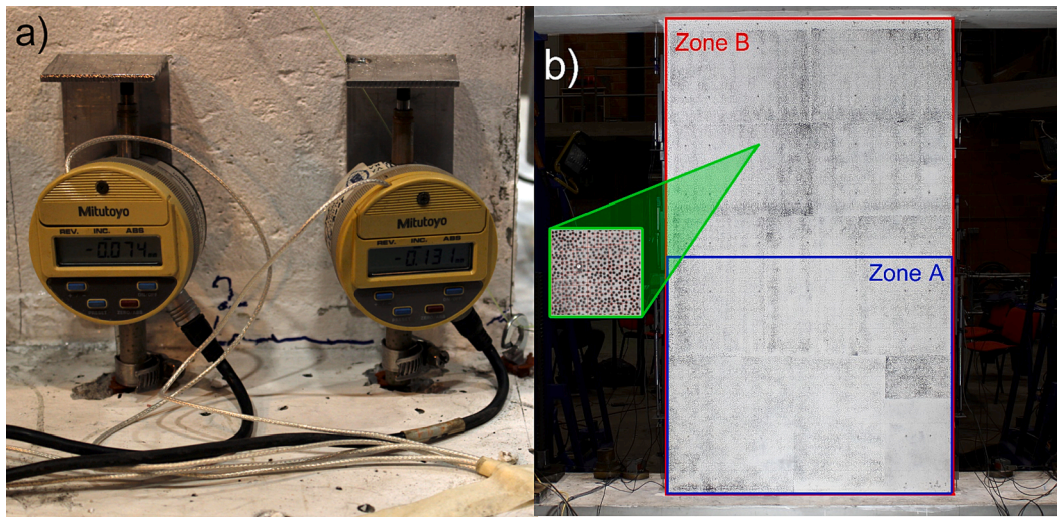


Fig. 2. (a) Mitutoyo digital micrometres at the base (b) speckle pattern in Zones A and B captured by the digital image correlation technique for specimen RC and RC + SMA respectively.

wall, one in the centre and four on each side along the wall length, to determine the vertical deformations caused by the expected large, primary crack between the foundation and the wall. The four sensors on each side at the base were distributed over the 300 mm confined boundary end (see Fig. 1a), spaced at 100 mm. These Mitutoyo sensors determine the displacement to the micrometre. A speckle pattern for digital image correlation (DIC) measurements was applied to the surface of each specimen (Fig. 2b) by a stencil to produce a random pattern of black dots with an approximate diameter of 4 mm. High resolution (12 Megapixel) cameras recorded monochrome (i.e., black and white) images at a frequency of 0.1 Hz (i.e., every 10 s) during testing. Two different zones captured the surface of the wall for the separate specimens (Fig. 2b): for the RC unit, a lens with a focal length of 25 mm was placed at 3.8 m from the wall, which limited the maximum height (from the base of wall) that could be captured by the camera. Zone A in Fig. 2b

for specimen RC corresponds to the full length of 1.2 m and a height of 1.3 m. For unit RC + SMA, Zone B was achieved, which captured the entire wall surface (i.e., length of 1.2 m and height of 2 m). The images were processed using vIC-2D (Correlated [20]), where the displacement and strain fields were exported into a binary data container format for MATLAB [51] to be analysed. The red circle markers in Fig. 3 indicate the cyclic peak displacement and force values corresponding to the DIC results that will be used later in this paper to derive the salient experimental observations.

3. Strain and crack distributions

This section presents strain and crack distribution results using the DIC data for the two wall units. More specifically, this section presents the strains distribution across the face of the walls in the form of heat

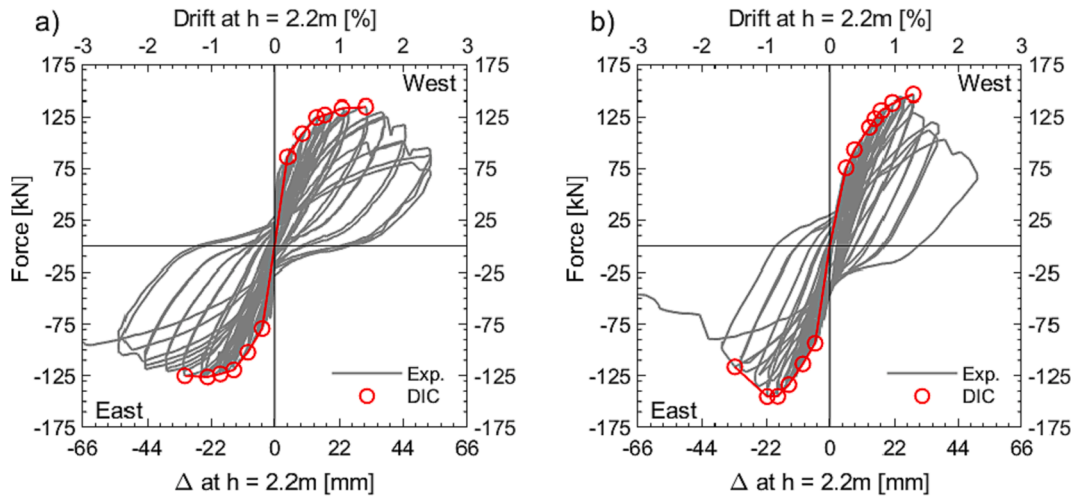


Fig. 3. Circles indicate the load steps captured by the digital image correlation (DIC) that were used for this investigation (a) unit RC + SMA and (b) unit RC.

maps. Furthermore, the principal strains from the DIC data are used to calculate crack patterns and crack widths, which are compared to the sketches documented during the experiments.

The longitudinal strain distributed across the face of the walls was determined from the DIC data and using a base length of 100 mm. This base length was the same as that used in Hoult & Beyer [38] to derive vertical strains from DIC data for RC U-shaped walls. The displacement field from the speckle pattern on the walls (Fig. 2b) does not capture the strain penetration into the foundation. This is particularly important for the two wall specimens investigated in this paper, which were both found to have a large, primary crack forming at the base that resulted in much of the plastic rotation occurring at the foundation level [5]. Thus, instead of relying solely on the DIC data, the micrometres (Fig. 2a) were used to estimate a more reliable value of the tension strain at the base, which included an approximate length of strain penetration into the foundation. The micrometres measured the vertical deformation of the wall at the base over a length of 60 mm, in addition to the vertical deformation from the strain and slip of the reinforcing bars below the foundation level. The vertical length in the foundation over which this additional deformation occurs is defined here as the strain penetration length (L_{sp}), which can be estimated using Equation (1) from Priestley et al. [61] for deformed longitudinal steel according to:

$$L_{sp} = 0.022f_{sy}d_{bl} \quad (1)$$

where f_{sy} is in MPa.

Using the mechanical properties for the steel bars in Almeida et al. [5] ($f_{sy} = 517$ MPa), the L_{sp} for the $d_{bl} = 12$ mm reinforcing steel was estimated to be 136 mm. Therefore, the longitudinal tensile strains at the base were calculated from the micrometres assuming that the vertical elongation is captured over a base length of 196 mm (i.e., 60 mm base length of the micrometres in addition to the L_{sp}). Given the paucity of research focusing on the yield penetration length of smooth SMA rebars, a L_{sp} of 140 mm is used for these bars, which corresponds to the length of the SMA bars into the foundation and terminating in the mechanical coupler, which in view of its bulky dimensions is considered to be fixed inside the foundation.

The resulting longitudinal strains are presented in Fig. 4 for unit RC + SMA in the form of a heat map at a drift of 0.4% and 1.4% in the East and West directions (according to the cardinal points that are given in Fig. 1a). A drift level of 1.4% drift corresponds to the peak strength attained by unit RC + SMA (see Fig. 3a). The reinforcement detailing of the wall is also superimposed in the heatmaps. Fig. 4 illustrates that the largest strain demands are concentrated at the base of the wall. While it is not immediately obvious in Fig. 4, due to the magnitude of the tensile strains at the foundation-wall level, some strain demands also

accumulate over some length across the section of the wall into web. At a drift of 1.4% to the East (Fig. 4d), large tensile strains can be observed in the web of the wall at approximately 250 mm from the base, i.e. within the SMA rebar region length of 270 mm. This type of strain distribution, with large demands reaching the web, could explain the steel rebar ruptures heard during testing [5]. Steel rebar rupture in the web section of the wall, prior to the SMA bars in the boundary ends, have also been reported in other testing on walls with similar detailing [2,45]. There are several reasons for this behaviour, including the light amount of steel used to reinforce the web region of the wall. However, it is believed that a significant cause for this accumulation of strain into the web is the large difference in axial stiffness between the SMA rebars used in the extremities of the wall and the conventional steel used in the web, due to the difference between the Young's modulus of the SMA and the steel, respectively 31 GPa and 205 GPa. This is an important point for design and assessment, which should be further explored.

For the specimen RC (Fig. 5), the longitudinal strains could only be calculated up to a wall height of 1200 mm due to the aforementioned DIC limitations during this unit test [5]. The resulting longitudinal strains in Fig. 5 for unit RC are presented for drift levels of 0.4% in both the East and West, 1.3% (West), and -1.0% (East). The strain distributions at 0.4% drift for this wall unit represent those prior to the formation of the foundation cone at -0.9% drift [5]. Drift levels of 1.3% and -1.0% correspond approximately to the peak strengths attained by RC unit in either of the directions (see Fig. 3a). At these two drift levels (1.3% and -1.0%), one can observe some, albeit limited, distributed inelastic strains up the height of the wall prior to a concentration of strain at the base crack, which ultimately resulted in failure of this specimen governed by rebar rupture. The strain heat maps for specimen RC at these drift levels are typical of a flexurally-governed wall, with large tension or compression strains being reached in the respective extreme fibre regions.

The documented crack patterns that were sketched by hand during testing are presented in Fig. 6. Previous research investigations have produced crack patterns of RC specimens from DIC strain fields [26,34,38]. A more recent research study developed a method for automated-crack detection and crack measurement (ACDM) using the DIC principal tensile strain data [29–30]. Using the ACDM procedure, crack patterns and salient crack widths were determined for specimens RC + SMA and RC in the two tested directions and for two drift levels. The large base crack that formed between the foundation and the wall for both specimens cannot be obtained from these analyses since it occurs along the bottom edge of the zone monitored by the DIC. As such, only the principal strains from the DIC data are used here, and not those derived from micrometres previously in this section. The high resolution

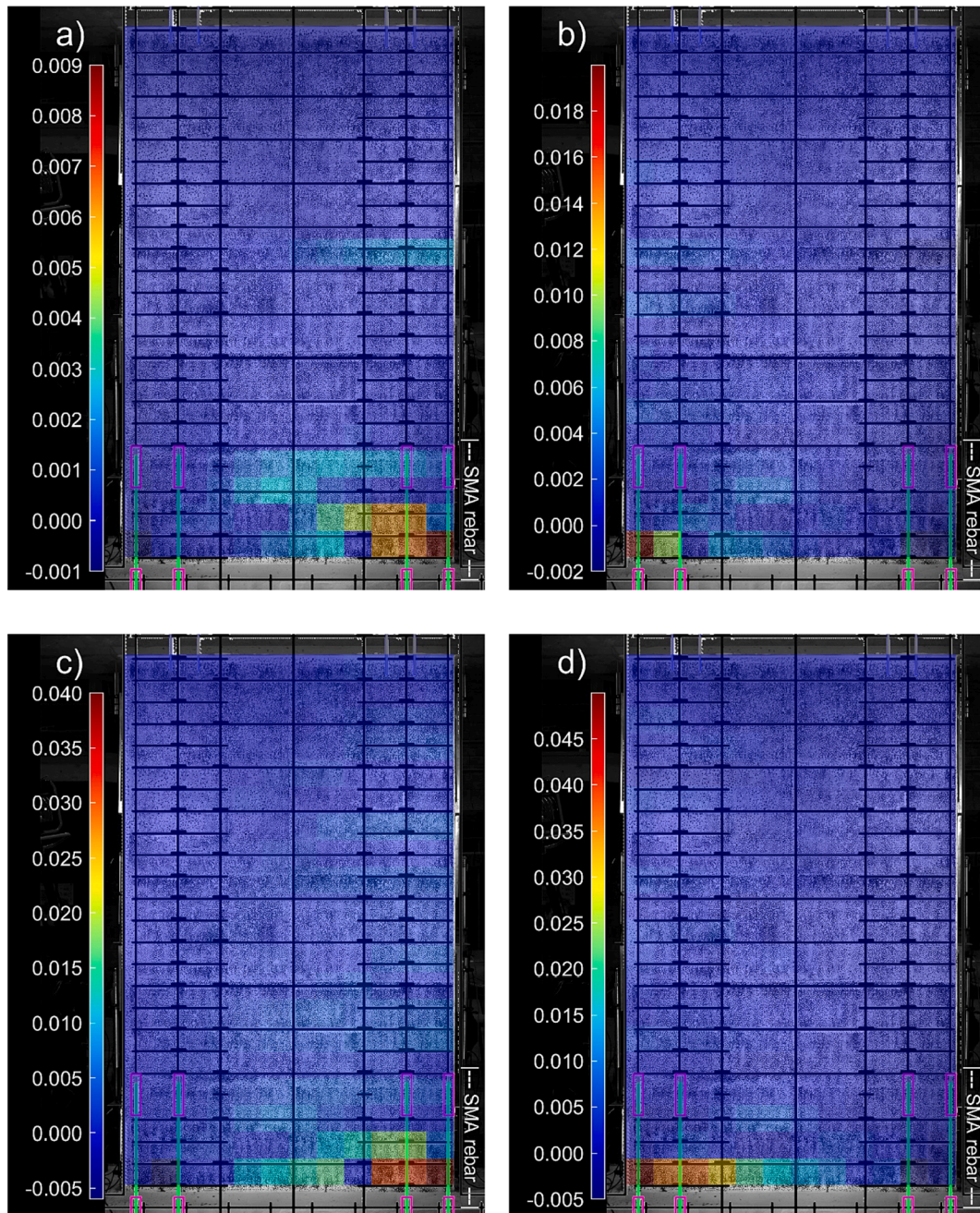


Fig. 4. Heat map of longitudinal strains of wall specimen RC + SMA using a base length of 100 mm at drift levels of (a) 0.4% West, (b) -0.4% East, (c) 1.4% West, and (d) -1.4% East (note the different scales).

(approximately $3.7 \text{ mm} \times 3.7 \text{ mm}$ grid) principal strains determined by vIC-2D (Correlated [20]) were used in the ACDM method to estimate the crack patterns and salient crack widths.

Fig. 7 illustrates the crack distributions using the ACDM procedure for specimen RC + SMA, which correspond well to that documented during testing illustrated in Fig. 6a. The largest crack widths estimated from the ACDM procedure are indicated in Fig. 7 (above each crack, in millimetres). For comparison, the manual crack measurements during testing are also indicated in Fig. 7 (below each crack in between parenthesis, in millimetres). The manual measurements were taken for the cracks in the extreme tension boundary region of the wall using an electronic crack measuring device, the values of which were only released in the digital laboratory notebook available in the online dataset [6]. Overall, the manual crack measurements taken during testing correspond reasonably well to those determined using the ACDM

method in Fig. 7. The small discrepancies between the crack measurements can be attributed to the two different methods used (i.e., human error) as well as slight variations in the locations at which the crack widths have been measured. As explained in the previous paragraph, the salient crack widths of the large base crack that formed during testing of the unit RC + SMA could not be calculated using the ACDM procedure. However, the digital laboratory notebook noted the largest crack width at the base to be 2.592 mm and 11.665 mm for drift levels of 0.4% and 1.4% (to the West) and 1.863 mm and 8.425 mm for drift levels of -0.4% and -1.4% (to the East); these crack widths are also indicated in Fig. 7. A large crack, which forms in both loading directions, appears to extend diagonally upward from the base in the extreme tension fibre region and across into the web of specimen RC + SMA. The crack patterns and estimated crack widths are consistent with a concentration of strain demands at the base of the wall. The small length of the SMA

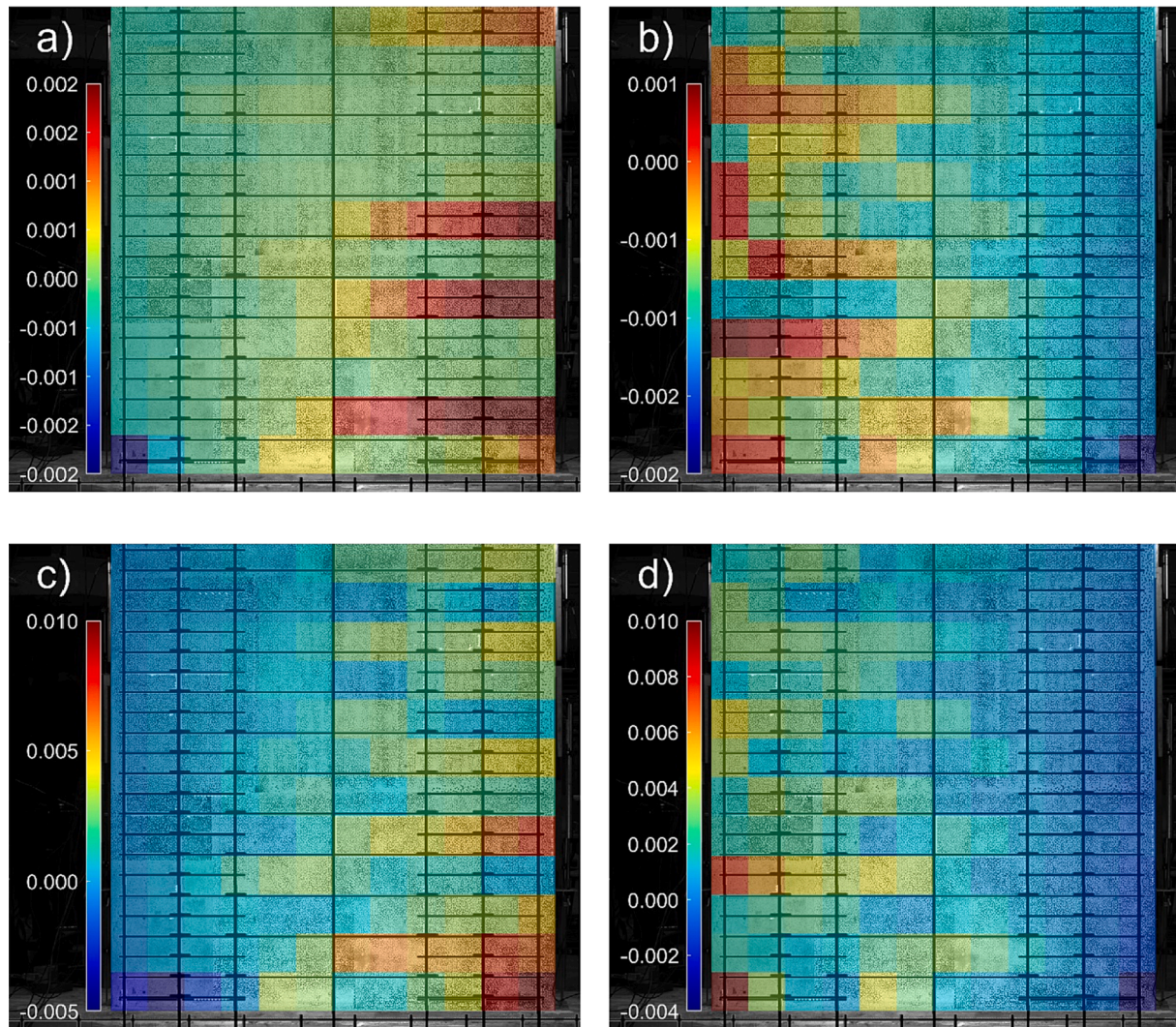


Fig. 5. Heat map of longitudinal strains of wall specimen RC using a base length of 100 mm at drifts levels of (a) 0.4% West, (b) -0.4% East, (c) 1.3% West, and (d) -1.0% East.

rebars, the use of mechanical couplers in the plastic hinge region, the smooth surface of the SMA bars (and corresponding reduced bond strength), and above all the capacity design principles employed to select the SMA rebars (as explained in Section 2), contributed to control the crack distribution. In particular, those factors limited the number of cracks and respective width above the SMA region at the base, until the yield strength of the longitudinal steel rebars was eventually reached. Before this point, an essentially flag-shaped force–displacement response was obtained, with strain demand concentrations at the base of the wall. The latter will be further explored in Sections 4 and 5.

Fig. 8 illustrates the crack distributions using the ACDM procedure for specimen RC (largest crack widths indicated above each crack, in millimetres). A reminder that only the lower 1200 mm of this wall specimen was captured using DIC techniques, and hence the top 800 mm of the illustrated wall has been shaded. For comparison, the manual crack measurements taken during testing [6] are also indicated in Fig. 8 (below each crack in between parenthesis, in millimetres). Near-horizontal cracking occurs for this wall specimen, which was initially noted in Almeida et al. [5], confirming the predominantly flexural behaviour of the structure. At larger drift levels, some inclined flexure-shear cracks can be observed high up the wall (Fig. 8b). It is noted that an overall comparable shear-flexure crack pattern between the two wall units can be observed for similar levels of large drifts above the base (see Fig. 7d and Fig. 8d); however, this apparent similarity reflects in fact a

pre-failure condition for the RC + SMA unit, and post-failure for the RC wall. As discussed later, the unexpected formation of a large foundation “cone” was partly responsible for the concentration of strains at the base of unit RC [5]. A more representative view on the behaviour of the units should therefore be searched at drift levels preceding this cone formation. For example, by comparing Fig. 7a and Fig. 8a, or Fig. 7c and Fig. 8c, i.e. at 0.4% drift West and East, one can see that – above the base – the number of cracks in the RC specimen is higher than for the unit RC + SMA, and simultaneously that the maximum crack width is roughly twice as large. In general, and although the RC wall plasticity concentrated primarily at the base crack for the larger drifts, the flexural cracks up the wall height (Fig. 8b,d) are large enough to make a contribution to the overall plastic rotation, which will be explored further in Section 5. At large enough drifts, the RC wall specimen loaded to the East (Fig. 8d) formed a large crack above the base in the web of the wall at a height of approximately 75 mm, or the spacing between the foundation and the first layer of shear reinforcement. The smaller amount of transverse reinforcement in the web with respect to the boundary elements (considering confinement reinforcement) can also help to explain the larger average crack spacing in the web. The crack patterns produced here from the strain maps correspond reasonably well to the sketches drawn manually during the experiment (Fig. 6b). It is also interesting to see the documented crack patterns higher up in the wall in Fig. 6b, which were not captured by the DIC due to the discussed restrictions for

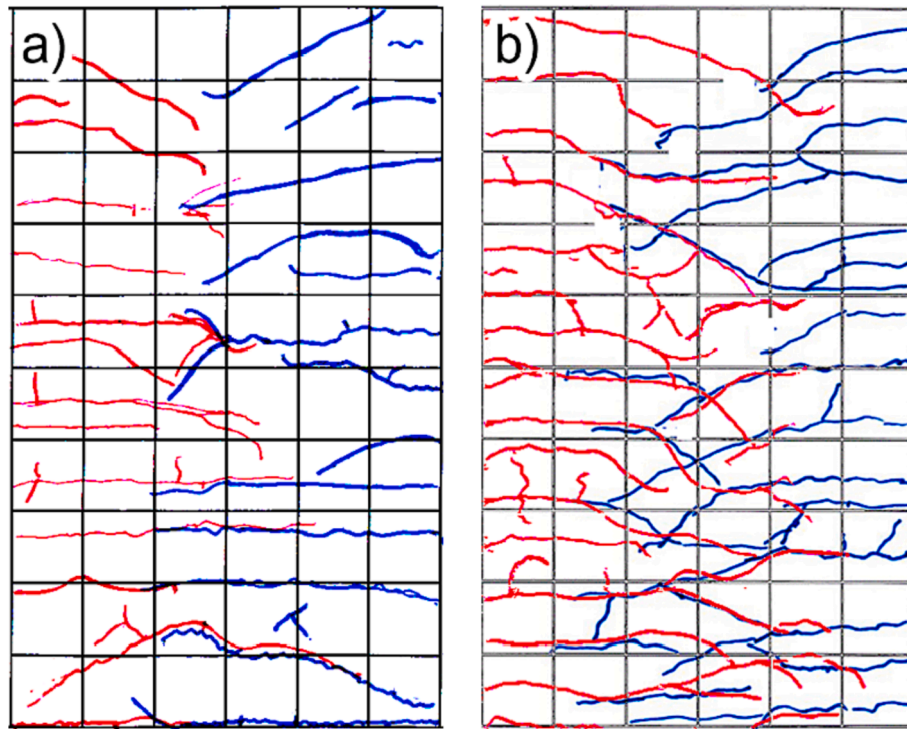


Fig. 6. Sketches of final crack patterns documented during testing [5]: (a) unit RC + SMA (b) unit RC.

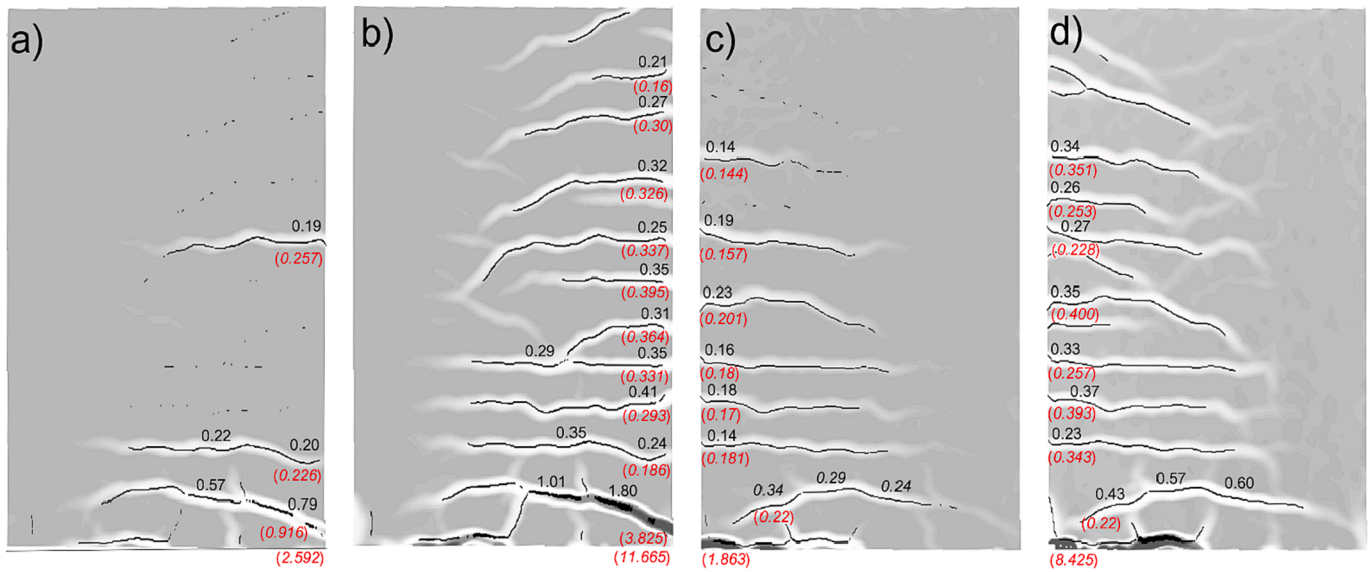


Fig. 7. Crack distributions for unit RC + SMA overlaying outlined principal strains (magnitudes not indicated) at drift levels of a) 0.4% (b) 1.4% (c) -0.4% and (d) -1.4% . Crack widths determined from the ACDM procedure are indicated above each main crack, whereas manual crack measurements are below the crack and in parenthesis (in millimetres, not to scale).

this specimen. The outermost micrometre in the tension fibre region was used to gage the approximate width of the primary base crack at the foundation level, indicated in Fig. 8. The vertical uplift from the micrometre was measured to be 9.93 mm at -1.5% to the East (Fig. 8d), which was the direction of the formation of the foundation cone; compare this to just 1.085 mm measured at the base at 1.0% drift to the West (Fig. 8b).

4. Strain profiles and euler-bernoulli hypothesis

A linear strain profile across the wall length is typically assumed in

the design of RC walls using displacement-based design and assessment procedures [70] and is inherent in moment-curvature estimates using a sectional analysis. The linear-strain hypothesis is also used to determine the flexural capacity of walls, which is compared with the demand obtained in the scope of classical force-based design methods. This assumption comes from the elementary theory of bending, or the Bernoulli-Euler assumption, that plane sections remain plane after bending. The application of this approximation is known to be only appropriate when there is no shear stress or for when the structural element has infinite shear stiffness [46]. This assumption is also not necessarily justifiable for non-rectangular sections (e.g., U-shaped),

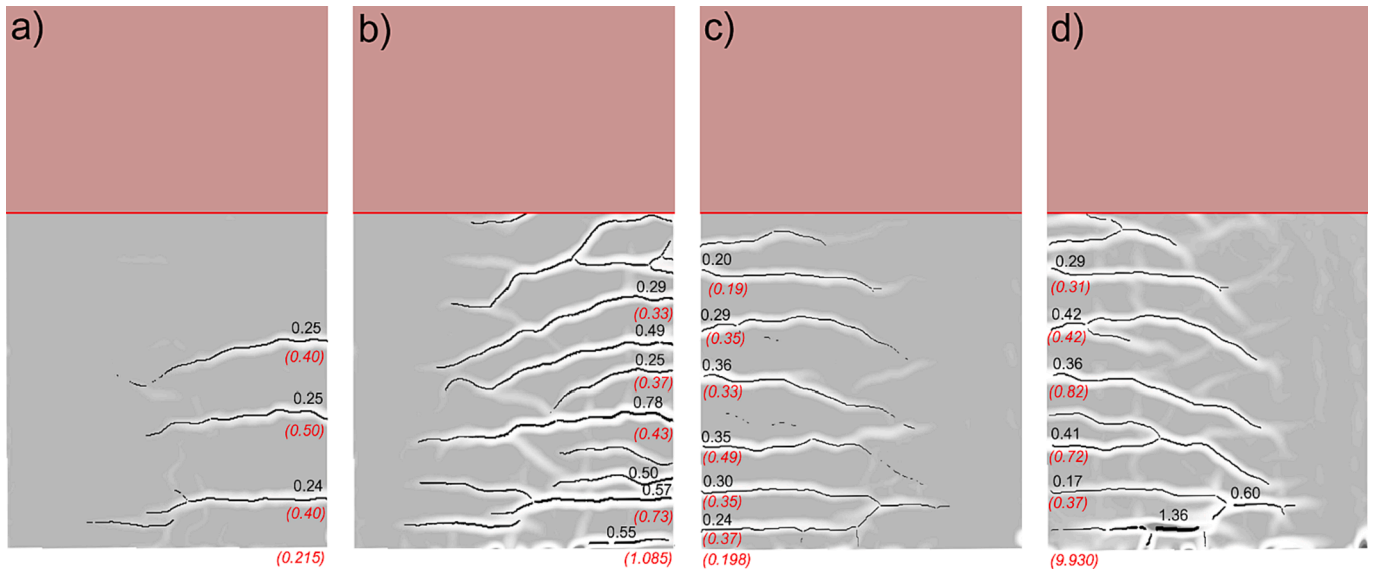


Fig. 8. Crack distributions for unit RC overlaying principal strains (magnitudes not indicated) at drift levels of (a) 0.4% (b) 1.0% (c) -0.4% and (d) -1.5%. Crack widths determined from the ACDM procedure are indicated above each main crack, whereas manual crack measurements are below the crack and in parenthesis (in millimetres, not to scale).

where the effects of shear lag can substantially influence the strain gradient across the wall section [19,38,40,56]. Even for slender flexurally-governed rectangular walls with relatively high shear span ratios, such approximation appears to hold only for elastic or relatively incipient inelastic material demands according to numerical simulations [4]. Such nonlinear strain gradients have been shown to occur experimentally rectangular walls, appearing to be more pronounced with higher concrete strengths [71]. Variation of the longitudinal reinforcement ratio along the wall length has been found to exaggerate this behaviour [22,23], particularly when walls have lumped reinforcement in the boundary ends [35]. With a view to continue improving the critical relation between material and member response, it is fundamental to investigate the strain gradient across the wall section. This is performed below at the wall base and at different heights, and for the first time when these two different reinforcing materials, conventional steel and SMAs, are used.

Using the longitudinal strains derived from both the DIC data and micrometres, Fig. 9a plots the distributed strain across the wall length

and at four different vertical distances up from the base: 0–100 mm, 100–200 mm, 200–300 mm, and 300–400 mm. The strains for specimen RC + SMA, at an in-plane drift of 1.4%, are used for this example. A fluctuation of strain at different heights can be observed in Fig. 9a. This fluctuation was also observed for the walls tested in Hoult & Beyer [38], albeit more pronounced due to the non-planar section and the absence of an axial load (i.e., no pre-compression to the wall). Some micrometres detached from the walls during the experiments, which explains the value of zero strain at an horizontal wall distance of 850 mm, marked with the dashed line in Fig. 9a. Therefore, the same method used in Hoult & Beyer [38] will be employed here, which uses the maxima tensile or compressive strain found within 400 mm vertical distance from the base and at each 100 mm increment across the wall length, resulting in the strain distribution in Fig. 9b. This method was found to be useful only for when large tensile strains could be observed in the web region at some height up from the foundation (as observable in Fig. 4 and discussed in Section 3). However, for most of the drift levels investigated here, the maximum strains at the base were found to be

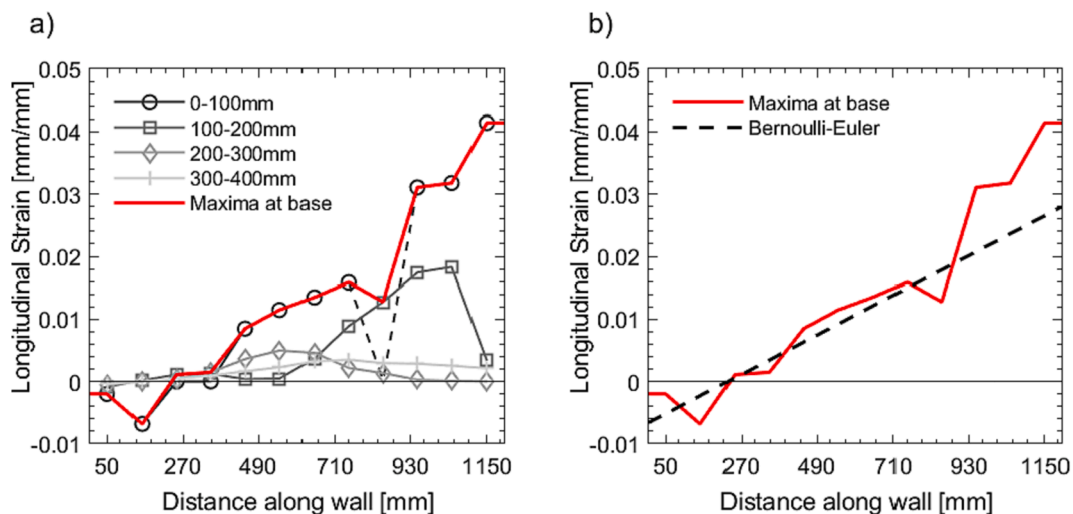


Fig. 9. RC + SMA wall at an in-plane drift (δ) of 1.4% (to the West): (a) strain distributions along different vertical distances from the base and (b) resulting strain distribution using the maximum (or minimum) strain values.

within the bottom 0–100 mm, as expected. It can be observed, from Fig. 9b, that the resulting strain distribution does not differ significantly from a linear Bernoulli-Euler profile. However, there is clearly a heightening of the tensile strain in the extreme tension fibre region, which does diverge from the linear strain gradient assumption. For this wall unit, it is likely that the difference in material stiffness of the lumped SMA rebars in the boundary end compared to that of the conventional steel used in the web is responsible for the observed strain gradient behaviour.

Fig. 10 presents the vertical strain profiles, obtained with the above method, for unit RC + SMA for a range of drift levels up to 1.4%. The yield strain of the reinforcing steel used in both wall units was measured as 0.0025 mm/mm [5], which is superimposed in Fig. 10 along the length of the wall where longitudinal steel bars were used (see Fig. 1a). A “pseudo-yield” strain of 0.01 mm/mm has also been indicated in Fig. 10 in the tension boundary end of the wall where SMA bars have been used. This pseudo-yield corresponds to the strain level at the end of the austenite phase of the SMA rebar [37]. First yielding of the outermost steel for unit RC + SMA (located approximately 350 mm from the tension wall edge) occurs at a drift ranging between 0.2 and 0.4%. Pseudo-yield is reached at a drift ranging between 0.4 and 0.6%, at which the nonlinear strain gradients for this wall are more pronounced, particularly towards the East (Fig. 10b). At these drift levels of 0.4–0.6% and to the East, there appears to be a spike in tensile strain in the extreme tension fibre region, where the SMA rebars are located, which does not conform to the linear strain profile assumption. Post-pseudo-yield, the strains gradients are clearly more consistent with a linear strain profile, although some nonlinearity still persists. Thus, these results indicate that, pre-pseudo-yield, the Bernoulli-Euler hypothesis, that plane sections remain plane, may not necessarily be justifiable for RC-SMA walls. Whether this greatly impacts moment–curvature estimates using a sectional analysis requires further experimental and numerical investigations.

Fig. 11 presents the vertical strain profiles at the base of unit RC, for a range of drift levels up to 1.5%. First yield occurs at drift levels of around 0.4–0.6%, with strains in the extreme tension fibre region reaching approximately 0.25%. Pre-yield, the strain profiles appear to be approximately linear. After yielding, the strain profiles deviate gradually into a more nonlinear distribution. This is particularly true for the RC wall at a drift of -1.5% (Fig. 11b), where there is a large rate of increase in the tensile strains occurring in the boundary end. A nonlinear distribution of strain post-yield was also observed in the planar RC walls investigated in Dashti et al. [23]. In this latter investigation, a considerable shift of the neutral axis at yielding and post-yielding stages of RC wall specimens caused a nonlinear strain profile, which was found to be

heavily dependent on the variation of the reinforcement ratio along the wall length. It is also worth noting that, for this wall unit, an unexpected foundation “cone” formed at a drift of -0.9% (to the East, Fig. 11b), wherein the failure surface “dived” several centimetres into the foundation. This resulted in a concentration of strains at the base crack and a non-ductile performance of the wall [5]. This foundation cone could provide another explanation for the peak observed in the bars in the extreme tension fibre region, which is only observable when pushing the wall to the East and is more pronounced after drifts of approximately -0.8% (Fig. 11b). In comparison to the strain gradients of the RC + SMA unit (Fig. 10), the strain profiles for the RC unit (Fig. 11) appear to be more linear at earlier drift levels, which is very likely a consequence of using an identical reinforcement material along the length of the wall, with the same elastic modulus and roughly uniformly distributed. The strain results here indicate that the assumption that plane sections remain plane after bending may only be applicable to RC sections with more uniform longitudinal reinforcement detailing. In particular, RC walls with low amounts of longitudinal reinforcement detailed in the web, in comparison to the boundary regions, are likely to have a more nonlinearized strain profile across the wall.

To further investigate the Bernoulli-Euler hypothesis for these wall units, the longitudinal strains calculated from the DIC and micrometre data were plotted across the wall length at different wall heights. Fig. 12 illustrates the variation of longitudinal strains for wall unit RC + SMA for two distinct drift levels towards the East: at -0.8% drift (Fig. 12a), where the wall unit had yielded and concrete had started to crush [5], and -1.4% drift (Fig. 12b), where the wall unit had reached its maximum strength and started to degrade. At these two drift levels, the variation of strains look very similar, although the magnitudes of the strains are different. While the magnitude of the longitudinal strains at the base crack vary marginally between the two drift levels (e.g., strain increases of approximately 13%, although accompanied by an extension along the section length), the strains above the base crack almost double. Conforming with the previous results in Section 3, most of the vertical strains appear to be concentrated at the base, which accounts for the strains into the foundation (i.e., strain penetration). Just above this region, the largest vertical strains appear to be in the web of the wall, as indicated previously in Section 3. The shape of maximum vertical strains experienced above the base crack of this unit appear to be bell-shaped across the length of the wall, with a peak in the web occurring at approximately 400 mm from the base. It is also interesting to note that the total height of the SMA rebars from the base, including the length into the mechanical coupler, is approximately 410 mm. As postulated in Section 3, it is likely that these unexpected strain gradients concentrating into the web of the wall are due to the difference in axial stiffness

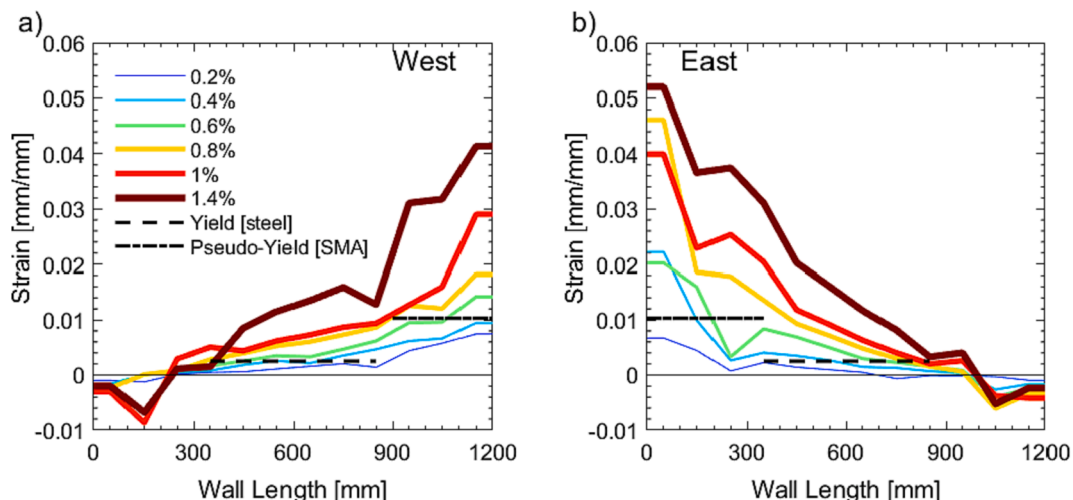


Fig. 10. Longitudinal (vertical) strains at the base of wall unit RC + SMA when being pushed in the (a) West direction and (b) East direction.

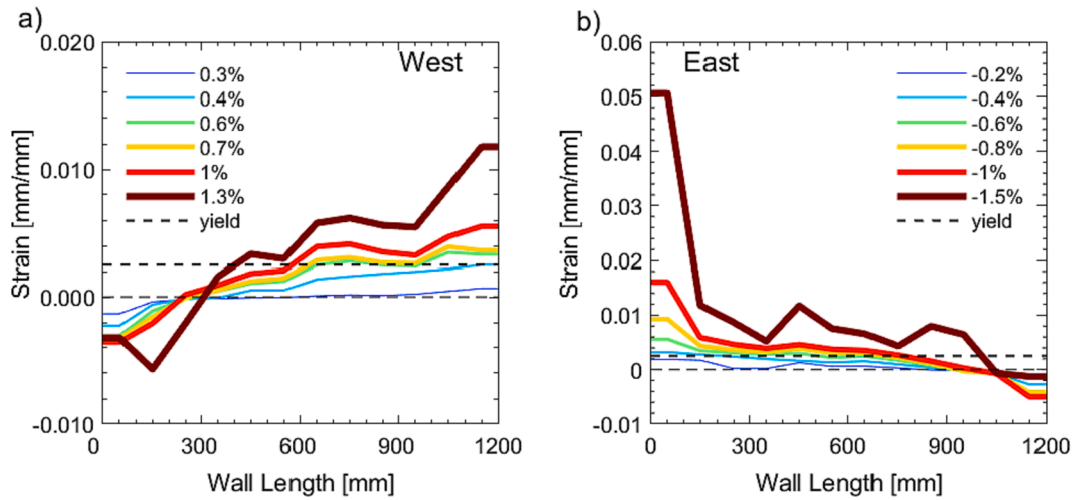


Fig. 11. Longitudinal (vertical) strains at the base of wall unit RC when being pushed in the (a) West direction and (b) East direction.

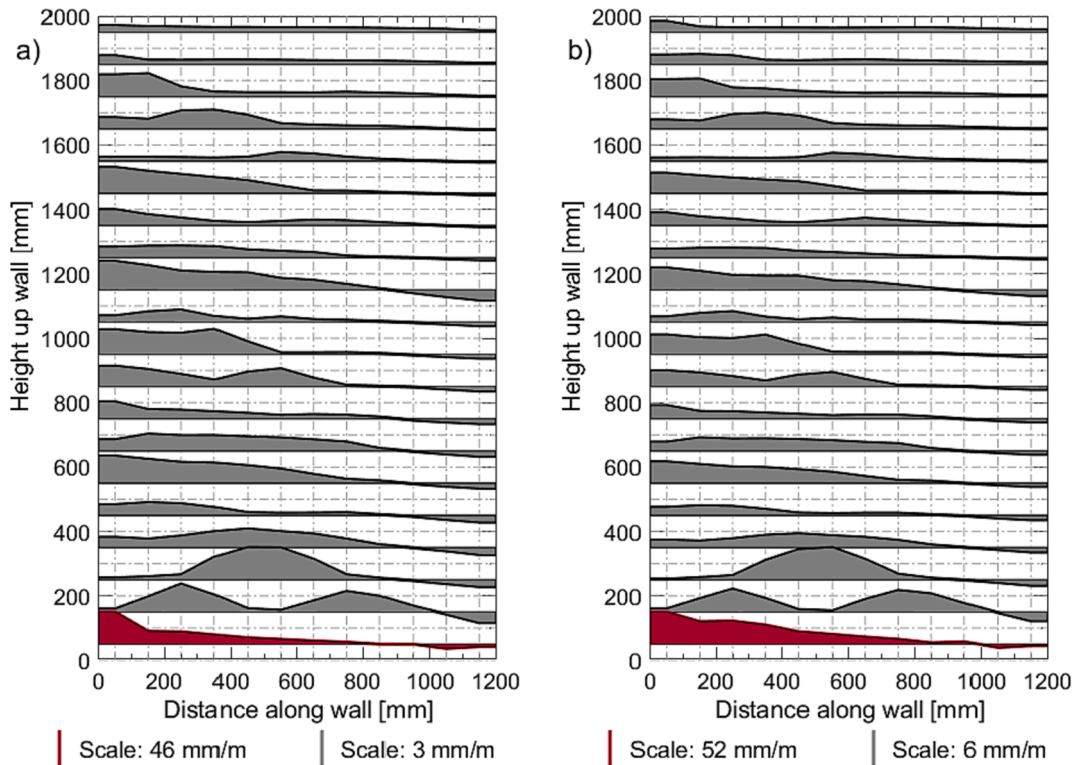


Fig. 12. Longitudinal strain profiles (or gradients) across the wall of unit RC + SMA at different heights being pushed towards the East (i.e., in the direction of right of page) at drifts of (a) -0.8% and (b) -1.4% .

between the SMA rebars and the conventional steel, as well as the light amount of steel used to reinforce the web of the wall. The presentation of these strain results in Fig. 12 further highlight the issue with assuming a linear strain profile across the wall section, which may be an inappropriate assumption for the design and analysis of RC-SMA walls.

Fig. 13 illustrates the variation of longitudinal strains for wall unit RC for two distinct drift levels towards the East: at -0.8% (Fig. 13a), which is the drift level prior to the formation of a large foundation cone and likely rebar fracture [5], and -1.5% drift (Fig. 13b). Unlike the strain variations for the RC + SMA unit (Fig. 12), which appeared to be consistent across the drift levels, there are some noticeable variations for the RC unit. For example, at drift levels prior to the onset of the unexpected foundation cone (i.e., $\delta < 0.9\%$), the vertical strains up the wall

height and across the length of the wall appear to be consistent with a flexurally governed wall with well distributed cracks. The magnitude of the vertical strains at the base crack of the RC unit are much less than the strains experienced at the base by the RC + SMA unit for the same level of drift of -0.8% (e.g., compare Fig. 13a with Fig. 12a). Furthermore, a more distinct compression zone can be observed in Fig. 13a for the RC unit in comparison to the RC + SMA unit (Fig. 12a), particularly at the base. At this level of drift ($\delta = -0.8\%$, Fig. 13a), the strain profiles across the wall and clear compression zone indicate that a simple linear strain gradient assumption could be applicable for this wall. Eventually, the formation of the foundation cone was accompanied by a sudden increase of the base crack width (at the tip of the cone), which also corresponded to a reduction of widths of the cracks above this primary base crack [5].

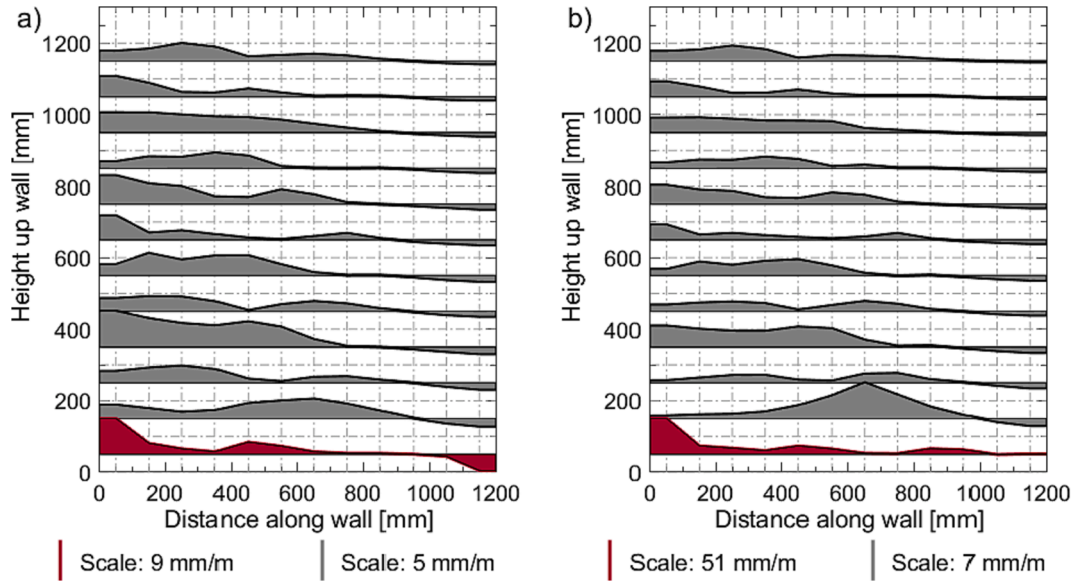


Fig. 13. Longitudinal strain profiles (or gradients) across the wall of unit RC at different heights being pushed towards the East (i.e., in the direction of right of page) at drifts of (a) -0.8% and (b) -1.5% .

This observation conforms with the strain variations in Fig. 13b, where the magnitude of the vertical strains at the base crack substantially increase, while the strains above this base crack at a drift of -1.5% appear to be of similar magnitude to those at -0.8% (in Fig. 13a). There also appears to be a spike of strain concentration in the lightly reinforced web after the formation of the foundation cone (Fig. 13b), which is consistent with the crack patterns presented in Section 3. The unintended consequence of the concentration of strains at the base, as indicated in Fig. 13b, question the validity of the assumption that plane section remains plane after bending for RC walls that behave in this non-ductile manner.

5. Equivalent plastic hinge lengths

A plastic hinge analysis (PHA) is one of the most widely used and simplest methods for calculating the displacement capacity of RC structures that are susceptible of being idealised as single-degree-of-freedom systems [4]. This method appeals to design engineers because (i) it is straightforward, (ii) it is based on familiar engineering quantities (e.g., moments and curvatures), and (iii) it has been partly incorporated into many seismic building standards [3]; Canadian Standards [13,16,17,41,68]. The PHA acknowledges that the top displacement of a cantilever wall is the summation of the deformation components primarily due to flexure, shear, and strain penetration, and where the ultimate displacement capacity is the summation of yield and plastic displacement contributions. In reality, the inelastic strains and curvatures will vary considerably up the height of the wall. However, in a PHA, the inelastic curvature is assumed to be uniform over a height equal to the plastic hinge length (L_p) [27]. Many empirical expressions are available in the literature for calculating L_p , which have been primarily derived from experimental or numerical investigations on beams, columns, highly confined walls, or bridge piers [12,44,55,60,59,61,63,70,72]. These expressions are generally limited to the range of design parameters that were used in each study. Based mostly on RC beam tests, an approximation for L_p as half the length of the wall (Equation (2)) was thought by many researchers to provide a safe and lower bound estimate [55,59,62,72].

$$L_p = 0.5L_w \quad (2)$$

Subsequently, Equation (2) was embedded in building standards to determine the design L_p for walls [41,68], or used to determine other

design expressions, such as defining when confinement is needed in wall boundaries (Canadian Standards [13]. Yet, numerous numerical studies [12,18,39,44] and experimental studies [1,25,48] have shown that Equation (2) can significantly overestimate the actual L_p of RC walls, which results in unconservative estimates of the displacement capacity of the structure. More recently, a study by one of the authors derived the plastic hinge length for 487 numerical and 234 experimental walls detailed with conventional steel to propose Equation (3) [36] for the assessment of RC walls detailed with steel rebars. While there are many empirical equations available for calculating the L_p , it is assumed that Equation (3) reflects the latest developments in this area of research.

$$L_p = r(0.13L_w + 18H_e^{0.6})(1 - 0.3ALR)(1 - 0.13\nu) \quad (3)$$

where r is the secondary cracking ratio ($=1.0$ for the walls, as in the present study), L_w is the wall length ($=1200$ mm), H_e is the effective height of the wall ($=4320$ mm), ALR is the axial load ratio ($=0.03$), and ν is the shear stress parameter defined with Equation (4).

$$\nu = \frac{V_{max}}{A_w \sqrt{f'_c}} \quad (4)$$

where V_{max} is the maximum base shear, A_w is the gross cross-sectional area of the wall ($=240,000$ mm²), and f'_c is the compressive strength of concrete ($=49.2$ MPa). Using the experimentally determined V_{max} (Fig. 3), the shear stress parameters (ν , Equation (4)) for specimen RC and RC + SMA is estimated to be 0.09 and 0.08, respectively. These low shear stress values reflect a planar wall governed by flexure.

A series of studies have also focused on deriving L_p for RC structures detailed with SMA rebars. Billah and Alam [11] numerically investigated the L_p of circular RC bridge piers detailed with SMAs for a range of design parameters, and derived Equation (5).

$$\frac{L_p}{L_w} = 1.05 + (0.25ALR) + 0.08 \left(\frac{H_e}{L_w} \right) + 0.0002f_{y-SMA} - 0.16\rho_l - 0.019f'_c - 0.24\rho_t \quad (5)$$

where f_{y-SMA} is the pseudo-yield strength of the SMA bars ($=319$ MPa), ρ_l is the longitudinal reinforcement ratio ($=0.85\%$), ρ_t is the transverse reinforcement ratio ($=0.67\%$), and f'_c is the concrete strength ($=49.2$ MPa).

More recently, a numerical study investigated the plastic hinge length of rectangular concrete columns reinforced with NiTi SMA bars

[47] and derived Equation (6).

$$\frac{L_p}{L_w} = 388 \rho_l^{1.367} \rho_t^{1.079} \left(\frac{f_{y-SMA}}{f_c} \right)^{1.559} \left(\frac{E_T}{E_A} \right)^{0.925} \left(\frac{s_t}{d_{bl}} \right)^{-0.614} \left(\frac{H_c}{L_w} \right)^{1.945} (1 - ALR)^{-0.153} \quad (6)$$

where E_T / E_A is the strain hardening ratio of the SMA rebar ($=0.08$) and s_t / d_{bl} is the transverse reinforcement spacing to longitudinal reinforcement diameter ratio ($=11.8$).

The authors have some reservations with regards to using Equations (5) and (6) for comparisons to the L_p values derived here for the RC + SMA wall unit. Firstly, these two expressions have been derived for RC columns, not walls, with a specific range of design values. Additionally, the methods employed to determine the inelastic strain profile, and thus the resulting equivalent L_p , involved a number of less refined assumptions that can limit the validity of the resulting predictive equations. Moreover, neither of these studies to derive Equation (5) and (6) modelled the bond behaviour between the concrete and the SMA rebars, which is likely to make a significant contribution to the performance of these structures. These equations also involve a large number of input parameters to derive an estimated L_p value.

Instead, what has been proposed by the authors is that the L_p of RC walls detailed with SMA bars at the base is simply equivalent to the SMA bar length used (i.e., $L_p = L_{b,SMA}$). The results from state-of-the-art finite element modelling of RC walls detailed with SMA rebars support this supposition [37]. Recent experimental testing on two RC columns were also found to have plastic hinge lengths of approximately 343 mm and 457 mm [47,58], which were equivalent to the two different SMA bar lengths used at their base. Thus, the authors believe that $L_{b,SMA}$, which excludes the overlap length in the mechanical couplers (see Fig. 1c), could be used to determine a conservative estimate of the L_p for RC walls detailed with smooth SMA rebars. More experimental testing on RC walls detailed with SMA rebars is needed to confirm the validity of this assumption (i.e., that $L_p = L_{b,SMA}$ for RC + SMA walls).

Curvature profiles are herein calculated with the intention of deriving the experimental L_p to compare with the expression above. The curvature profiles for units RC + SMA and RC are given in Fig. 14 for a range of significant drift levels. These curvatures were determined from the strain values in the extreme compression and tension fibre regions using the DIC and Mitutoyo micrometre data. The Bernoulli-Euler principle (i.e., linear strain profile through the section) was assumed in these calculations. As observed in Section 4, this assumption is not always necessarily justifiable, however, for simplicity, a linear strain profile is assumed in this section for calculation of the curvature profiles.

The nominal yield curvature (Φ_y) profile is superimposed in the following figures, where a linear yield curvature profile up the wall

height is assumed [61]. For simplicity, the curvature at first yield (Φ'_y) was calculated using a sectional analysis for the RC unit [8], and thus Φ_y at the base is found by linear extrapolation (e.g., $\Phi_y = \frac{\Phi'_y M_u}{M'_y}$, where M'_y and M_u are the first yield and ultimate moments, respectively). Furthermore, for the RC + SMA unit (Fig. 14a), a nominal “pseudo-yield” line is superimposed, which was obtained through the method similar to the one indicated above for the RC specimen cross-section. As defined in Section 4, the “pseudo-yield” here corresponds to the strain at the end of the austenite phase of the SMA rebar [37], which approximately corresponds to the start of a plateau in the stress-strain relation. Fig. 14a shows that the nominal “pseudo-yield” for the RC + SMA specimen was attained very early on, at drifts between 0.2 and 0.4%, which appears to be visually consistent with the force-displacement curve in Fig. 3a and the results in Section 4 (Fig. 10). Unlike the RC specimen, which showed some distributed plasticity up the wall height prior to the premature failure (see Section 3), the curvature profiles for RC + SMA specimen show a concentration of the curvature at the base of the wall with very limited curvature demand higher up in the wall.

The curvature profiles for the RC unit (Fig. 14b) show that yield is attained at a drift of approximately 0.4%, consistent with the observations made in Section 3, after which the curvature varies considerably up the wall height. At a drift of approximately 1.4%, the RC specimen to the West shows spread of plasticity occurring up to heights of around 1.2 m from the base (i.e., the maximum height captured by the DIC cameras for this specimen). Conversely, to the East at 1.4% drift, the plastic curvatures are observed to concentrate at the base of the wall in Fig. 14b. This observation is consistent with the preliminary summary of experimental results in Almeida et al. [5], where an increase in the base crack width accompanied by a small reduction of the widths of the cracks above, could be observed for this specimen after 0.9% drift. These curvature profile observations in Fig. 14 are consistent with the strain heat maps presented in Fig. 4 and Fig. 5 for the RC and RC + SMA specimens, respectively.

The equivalent plastic hinge length (L_p) can be determined from the experimental results by assuming that the plastic rotation (θ_p) is equivalent to an area with height of L_p and width Φ_p (Equation (7)):

$$L_p = \frac{\theta_p}{\Phi_u - \Phi_y} = \frac{\theta_p}{\Phi_u - \Phi_y} \quad (7)$$

where Φ_u is the curvature at the ultimate limit state and θ_p is the plastic rotation.

The curvature distribution up the height of a cantilever wall can be used to determine θ_p . As is well known from the first moment area theorem, the rotation is equal to the area under the curvature distribution. The plastic rotation can hence be determined using Equation (8).

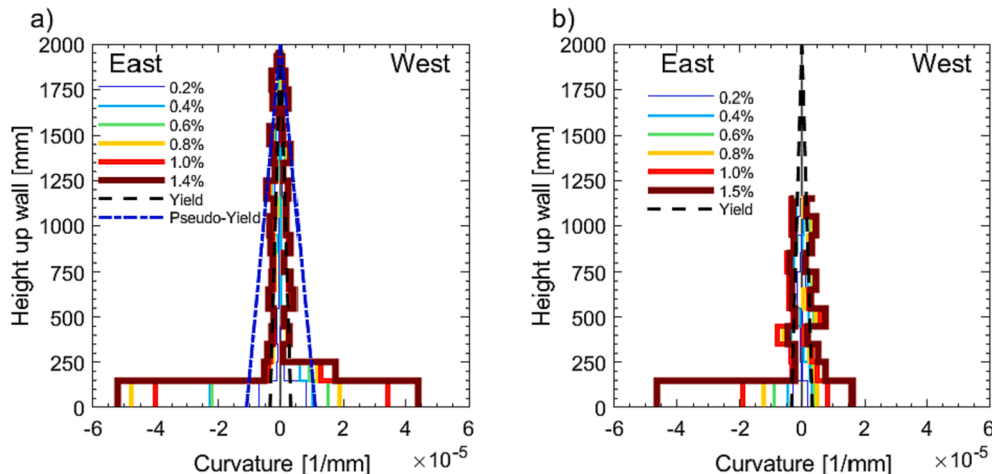


Fig. 14. Curvature distributions (a) unit RC + SMA and (b) unit RC.

$$\theta_p = \int_0^H |\Phi(x)| - \Phi_y(x) dx \quad (8)$$

where $\Phi(x)$ is the curvature found at a height of x along the cantilever wall, $\Phi_y(x)$ is the curvature at a height of x when the nominal yield curvature Φ_y is attained at the wall base, and H is the point of contraflexure (i.e., the application height of the actuators for these experimental units). Only curvatures $\Phi(x)$ that are larger in absolute value than Φ_y are considered in the integral above. It is also worth emphasising, as previously discussed, that a linear yield curvature profile is assumed and depicted in Fig. 14a,b. Furthermore, for calculating the L_p estimates of the RC + SMA unit, the nominal pseudo-yield curvature is substituted in Equation (8) for the Φ_y .

Fig. 15 plots the equivalent plastic hinge lengths (L_p) as a function of drift for units RC + SMA and RC, which were derived from the curvature profiles in Fig. 14 and Equations (7) and (8). The L_p results for specimen RC + SMA, shown in Fig. 15a, are consistent throughout the range of drifts. This is indicative of a concentration of strain demands at the base of the wall unit throughout the drifts imposed during testing, where the increase in magnitude of plastic curvature at the base is consistent with that of the plastic rotation (Equation (7)). It is likely that many factors have resulted in the concentration of strain demands at the base for this wall unit, including the small SMA bar length used, as well as the surface condition of the SMA bars and corresponding reduced bond strength [53]. The results in Fig. 15a further substantiate that a L_p equivalent to the SMA bar length used (i.e., $L_p = L_{b,SMA}$) can provide a reasonable estimate. The expression from Billah and Alam [11] (Equation (5)) also predicts a reasonable and slightly conservative estimate of the L_p for this wall unit in comparison to the experimentally derived values. However, it is interesting to note that numerical research previously conducted by the authors indicated that the shear span had no effect on the resulting L_p for flexurally governed RC walls detailed with SMA bars at the base [37]. In contrast, if the wall in question had, say, double the shear span, the L_p estimate using Equation (5) from Billah and Alam [11] would more than double. And so, while Equation (5) from Billah and Alam [11], derived for RC columns, has shown to produce a reasonable prediction for this wall unit, it is not clear if this expression would be accurate for other mid- or high-rise walls with SMA bars used at the base. More experimental results are needed to further verify these findings before any recommendations can be made with confidence. The expression from Lee et al. [47] (Equation (6)) estimated an unrealistically low L_p of just 60 mm, which is possibly a result of being specifically derived for RC columns and not walls.

The experimentally derived L_p values for the RC unit in Fig. 15b

appear to decrease as a function of drift, before starting to plateau when reaching the 0.9% drift. This is consistent with previous experimental results that have found the value of L_p of RC walls (detailed with conventional steel) generally decrease asymptotically as a function of drift prior to a plateau [9,25,38]. The L_p values plateau towards magnitudes less than what is predicted by the two relevant L_p expressions (Equation (2) and Equation (3)), particularly for those estimated in the East direction. However, there are several points to consider to understand this difference. Firstly, the DIC data for this specimen, used to derive the strain and curvature profiles, was restricted to a height of 1200 mm from the base of the wall (Fig. 14b). It is probable that some inelastic strains (and consequent plastic rotation) were not captured above this height, which would have resulted in larger plastic hinge lengths. In fact, the documented crack distributions in Fig. 6b support this supposition. Furthermore, the reported hysteresis shape of the RC unit was thinner than usual for RC members after the formation of the large base crack and foundation crack (i.e., after a drift of 0.9% is achieved, indicated in Fig. 15b), indicative of a restrained spread of plasticity [5]. Thus, it is likely that this wall had the potential for a larger L_p , corresponding to a better distribution of strains and curvatures up the wall height, but this was prevented due to the premature damage observed. The foundation cone that formed when pushing the RC wall unit towards the East could also explain the smaller L_p values in comparison to those obtained for the West direction in Fig. 15b for the entire drift range.

6. Conclusions

New experimental research findings were presented for the seismic performance of planar RC walls detailed with shape memory alloy rebars using digital image correlation techniques. They were employed to investigate the applicability of the Euler-Bernoulli hypothesis by computing strain distributions and the plastic hinge length. The two wall specimens were tested at the LEMSC laboratory of the iMMC at UCLouvain, Belgium. The initial test observations for the two wall specimens were described in some detail in Almeida et al. [5]. A summary of the new research findings herein investigated are given below:

- The specimen detailed with shape memory alloys was found to concentrate strains at the base throughout the entire drift range until failure, in contrast to the unit fully detailed with steel rebars, which was in agreement with the design approach and allowed the spread of plasticity above the base. Notably, some concentration of large tensile strains in the lightly reinforced web of the wall was also

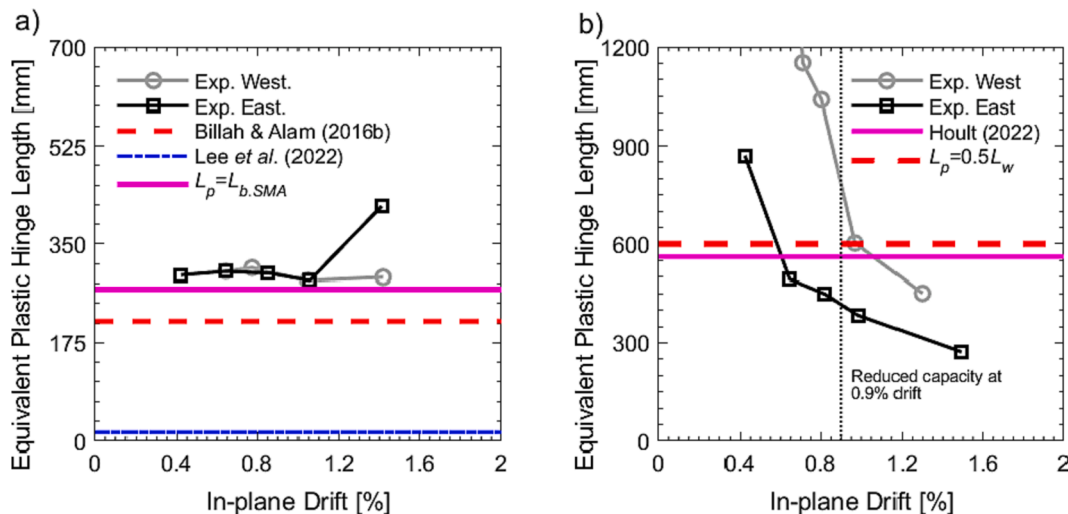


Fig. 15. Equivalent plastic hinge lengths derived from experimental ("Exp.") curvature profiles, and some expressions available in the literature for specimen (a) RC + SMA and (b) RC.

observed, which can be related with the large difference of the Young's modulus between SMA and steel rebars. Further research needs to be carried out to investigate this important point for design and assessment.

- The crack distribution and crack widths further confirmed the concentration of strains at crack locations close to the base of the walls. A large crack in the web for both specimens was likely due to the light amount of steel used in the web, both longitudinal and transverse, and emphasises the need for careful detailing in this area.
- Overall, the Bernoulli-Euler assumption, 'plane sections remain plane', reasonably simulated the near-linear strain profiles across the section for both specimens. However, the wall section was not very long, which may have limited the deviation of the strain profile from linearity, which has been pointed out in other studies. Some nonlinear branches of the post-yield strain profiles across the section could be observed, particularly for the specimen detailed with steel. The RC wall unit detailed with SMA rebars experienced a spike in tensile strains in the boundary ends, which did not conform with the linear strain profile assumption. This "spike" in tensile strain was likely to be due to the large difference in axial stiffness between the SMA rebars used in the extremities of the wall and the conventional steel used in the web, which may be an important consideration for the future design and assessment of these types of structures – it is recommended that this be further explored in future research.
- The equivalent plastic hinge length of the specimen detailed with shape memory alloys was near constant throughout the drift range. A plastic hinge length value equal to the shape memory alloy bar length used at the base was found to reasonably predict the experimentally observed values, further substantiating other recent research. This is a useful finding for engineering practice, which should be further confirmed by future experimental and numerical investigations varying the reinforcement detailing of the SMA rebars in the wall.
- The equivalent plastic hinge length computed for the specimen detailed solely with steel was found to be less than the predicted length as found by other common or state-of-the-art proposals. It is likely that the unexpected and premature formation of a large base crack for this specimen restricted the growth of plasticity up the height of the wall from the base and ultimately resulted in a reduced plastic hinge length.

7. Numerical data

The dataset from the research investigation undertaken in this paper can be downloaded from the publicly accessible platform Dataverse, <https://doi.org/10.14428/DVN/EB9KYW>. The structure of the data folders is described in the report 'Data-organization_Hoult_and_Almeida_2022.pdf', also available for download. The following data is provided: (1) Digital Image Correlation output files (2) MATLAB processing scripts to reproduce results and figures. This readily available dataset allows transparency of the data and files that were used to compile the results in this paper.

CRediT authorship contribution statement

Ryan Hoult: Conceptualization, Methodology, Software, Validation, Investigation, Data curation, Writing – original draft. **João Pacheco de Almeida:** Conceptualization, Validation, Writing – review & editing, Supervision, Project administration.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

The dataset from the research investigation undertaken in this paper can be downloaded from the publicly accessible platform Dataverse, DOI: 10.14428/DVN/EB9KYW

Acknowledgements

This work was supported by the Fonds de la Recherche Scientifique – FNRS under Grant n° F.4501.21.

References

- [1] Aaleti S, Dai H, Sritharan S. Ductile design of slender reinforced concrete structural walls. In: Paper presented at the Proceedings of the 10th National Conference in Earthquake Engineering; 2014.
- [2] Abdulridha A, Palermo D. Behaviour and modelling of hybrid SMA-steel reinforced concrete slender shear wall. *Eng Struct* 2017;147:77–89. <https://doi.org/10.1016/j.engstruct.2017.04.058>.
- [3] ACI. Building code requirements for reinforced concrete. ACI 318-89. In: Detroit: American Concrete Institute; 1989. p. 353.
- [4] Almeida JP, Tarquini D, Beyer K. Modelling approaches for inelastic behaviour of RC walls: multi-level assessment and dependability of results. *Arch Comput Methods Eng* 2016;23(1):69–100.
- [5] Almeida JP, Steinmetz M, Rigot F, de Cock S. Shape-memory NiTi alloy rebars in flexural-controlled large-scale reinforced concrete walls: experimental investigation on self-centring and damage limitation. *Eng Struct* 2020;220:110865.
- [6] Almeida J, Steinmetz M, Rigot F, et al. Shape-memory NiTi alloy rebars in flexural-controlled large-scale reinforced concrete walls: experimental investigation on self-centering and damage limitation. Dataverse, UCLouvain 2020. <https://doi.org/10.14428/DVN/2QBQVJ>.
- [7] Andrawes B, Shin M, Wierschem N. Active confinement of reinforced concrete bridge columns using shape memory alloys. *J Bridge Eng* 2010;15(1):81–9.
- [8] Bentz EC, Collins MP. RESPONSE-2000: Reinforced Concrete Sectional Analysis using the Modified Compression Field Theory. 1.9.6 (beta) ed. University of Toronto; 2016.
- [9] Beyer K. Seismic design of torsionally eccentric buildings with U-shaped RC Walls. (PhD). ROSE School, (ROSE-2008/OX); 2007.
- [10] Billah AM, Alam MS. Performance-based seismic design of shape memory alloy-reinforced concrete bridge piers. I: Development of performance-based damage states. *J Struct Eng* 2016;142(12):04016140.
- [11] Billah AM, Alam MS. Plastic hinge length of shape memory alloy (SMA) reinforced concrete bridge pier. *Eng Struct* 2016;117:321–31. <https://doi.org/10.1016/j.engstruct.2016.02.050>.
- [12] Bohl A, Adebare P. Plastic hinge lengths in high-rise concrete shear walls. *ACI Struct J* 2011;108(2):148–57.
- [13] Canadian Standards Association. CSA A23.3-04. Design of concrete structures for buildings. Toronto, ON, Canada; 2004. p. 258.
- [14] Cardone D, Dolce M, Ponzio FC. The behaviour of SMA isolation systems based on a full-scale release test. *J Earthquake Eng* 2006;10(06):815–42.
- [15] CEN. Eurocode 8: Design of structures for earthquake resistance. Part 1: general rules, seismic actions and rules for buildings. In: E. S. E. 1998-1:2004, editor. In. Brussels, Belgium; 2004. Comité Européen de Normalisation.
- [16] CEN. Eurocode 8: Design of structures for earthquake resistance. Part 3: Assessment and Retrofitting of Buildings. EN 1998-3:2005. Brussels (Belgium): European Committee for Standardization; 2005a.
- [17] CEN. Eurocode 8: Design of structures for earthquake resistance. Part 2: Bridges. EN 1998-2:2005. Brussels (Belgium): European Committee for Standardization; 2005b.
- [18] Cheng X, Zhang H. Numerical modelling of reinforced concrete slender walls subjected to coupled axial tension-flexure. *Int J Concr Struct Mater* 2021;15(1):33. <https://doi.org/10.1186/s40069-021-00471-y>.
- [19] Constantini R, Beyer K. Behaviour of U-shaped RC walls under quasi-static cyclic diagonal loading. *Eng Struct* 2016;106:36–52. <https://doi.org/10.1016/j.engstruct.2015.10.018>.
- [20] Correlated Solutions. VIC-2D (Version 7); 2019. Available from: <http://www.correlatedsolutions.com>.
- [21] Cortés-Puentes L, Zaidi M, Palermo D, Dragomirescu E. Cyclic loading testing of repaired SMA and steel reinforced concrete shear walls. *Eng Struct* 2018;168:128–41.
- [22] Dashti F, Dhakal RP, Pampanin S. Evolution of out-of-plane deformation and subsequent instability in rectangular RC walls under in-plane cyclic loading: Experimental observation. 2018a;47(15):2944–64. doi:10.1002/eqe.3115.
- [23] Dashti F, Dhakal RP, Pampanin S. Inelastic strain gradients in reinforced concrete structural walls. In: Paper presented at the 16th European conference on earthquake engineering. Thessaloniki, Greece; 2018b.
- [24] Dazio A. Residual displacements in capacity designed reinforced concrete structures. In: Paper presented at the The 13th world conference on earthquake engineering, Vancouver, BC, Canada; 2004.
- [25] Dazio A, Beyer K, Bachmann H. Quasi-static cyclic tests and plastic hinge analysis of RC structural walls. *Eng Struct* 2009;31(7):1556–71. <https://doi.org/10.1016/j.engstruct.2009.02.018>.

- [26] Destrebecq J-F, Toussaint E, Ferrier E. Analysis of cracks and deformations in a full scale reinforced concrete beam using a digital image correlation technique. *Exp Mech* 2011;51(6):879–90.
- [27] Dhakal RP, Fenwick RC. Detailing of plastic hinges in seismic design of concrete structures. *ACI Struct J* 2008;105(6). <https://doi.org/10.14359/20102>.
- [28] El-Borgi S, Neifar M, Jabeur MB, Cherif D, Smaoui H. Use of copper shape memory alloys in retrofitting historical monuments. *Smart Struct Syst* 2008;4(2):247–59.
- [29] Gehri N, Mata-Falcón J, Kaufmann W. Automated crack detection and measurement based on digital image correlation. *Constr Build Mater* 2020;256: 119383. <https://doi.org/10.1016/j.conbuildmat.2020.119383>.
- [30] Gehri N, Mata-Falcón J, Kaufmann W. Refined extraction of crack characteristics in large-scale concrete experiments based on digital image correlation. *Eng Struct* 2022;251:113486. <https://doi.org/10.1016/j.engstruct.2021.113486>.
- [31] Goldsworthy HM. Lessons on building design from the 22 February 2011 Christchurch earthquake. *Aust J Struct Eng* 2012;13(2):159. <https://doi.org/10.7158/S11-136.2012.13.2>.
- [32] Gonzalez R, Stephens M, Toma C, Elwood K, Dowdell D. Post-earthquake Demolition in Christchurch, New Zealand: a case-study towards incorporating environmental impacts in demolition decisions. In: *Advances in Assessment and Modeling of Earthquake Loss*; 2021. p. 47–64.
- [33] Holden T, Restrepo J, Mander JB. Seismic performance of precast reinforced and prestressed concrete walls. *J Struct Eng* 2003;129(3):286–96. doi: 10.1061/(ASCE)0733-9445(2003)129:3(286).
- [34] Hoult NA, Dutton M, Hoag A, Take WA. Measuring crack movement in reinforced concrete using digital image correlation: overview and application to shear slip measurements. *Proc IEEE* 2016;104(8):1561–74.
- [35] Hoult R. Minimum longitudinal reinforcement requirements for boundary elements of limited ductile walls for AS 3600. *Electron J Struct Eng* 2017;17(1):43–52.
- [36] Hoult R. A universal plastic hinge length for reinforced concrete walls. *ACI Struct J* 2022.
- [37] Hoult RD, de Almeida JP. Residual displacements of reinforced concrete walls detailed with conventional steel and shape memory alloy rebars. *Eng Struct* 2022; 256:114002.
- [38] Hoult R, Beyer K. RC U-shaped walls subjected to in-plane, diagonal, and torsional loading: New experimental findings. *Eng Struct* 2021;233:111873. <https://doi.org/10.1016/j.engstruct.2021.111873>.
- [39] Hoult R, Goldsworthy H, Lumantarna E. Plastic hinge length for lightly reinforced rectangular concrete walls. *J Earthquake Eng* 2018;22(8):1447–78. <https://doi.org/10.1080/13632469.2017.1286619>.
- [40] Hoult RD. Shear lag effects in reinforced concrete C-shaped walls. *J Struct Eng* 2019;145(3):04018270. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0002272](https://doi.org/10.1061/(ASCE)ST.1943-541X.0002272).
- [41] ICBO. International council of building officials, 1997 Uniform Building Code, Vol. 2. In: Whittier, CA; 1997.
- [42] Indirli M, Castellano MG. Shape memory alloy devices for the structural improvement of masonry heritage structures. *Int J Arch Heritage* 2008;2(2): 93–119.
- [43] Kawashima K. Seismic design and retrofit of bridges. *Bull New Zealand Soc Earthquake Eng* 2000;33(3):265–85.
- [44] Kazaz I. Analytical study on plastic hinge length of structural walls. *J Struct Eng* 2013;139(11):1938–50.
- [45] Kian MJT, Cruz-Noguez C. Reinforced concrete shear walls detailed with innovative materials: seismic performance. *J Compos Constr* 2018;22(6): 04018052. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000893](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000893).
- [46] Kwan AKH. Shear lag in shear/core walls. *J Struct Eng* 1996;122(9):1097–1104. doi: 10.1061/(ASCE)0733-9445(1996)122:9(1097).
- [47] Lee CS, Choi E, Jeon J-S. Estimating the plastic hinge length of rectangular concrete columns reinforced with NiTi superelastic shape memory alloys. *Eng Struct* 2022;252:113641.
- [48] Lu Y, Henry RS, Gultom R, Ma QT. Cyclic testing of reinforced concrete walls with distributed minimum vertical reinforcement. *J Struct Eng* 2016;04016225.
- [49] Maciel M, Palermo D, Abdulridha A. Seismic response of SMA reinforced shear walls. *Special Topics Struct Dyn* 2016;6:185–92. https://doi.org/10.1007/978-3-319-29910-5_19.
- [50] Marquis F, Kim JJ, Elwood KJ, Chang SE. Understanding post-earthquake decisions on multi-storey concrete buildings in Christchurch, New Zealand. *Bull Earthquake Eng* 2017;15(2):731–58. <https://doi.org/10.1007/s10518-015-9772-8>.
- [51] Mathworks. MATLAB R2021a Update 3 (9.10.0.1684407) 64-bit (win64). 2021. Available from: <https://www.mathworks.com/products/matlab.html>.
- [52] McCormick J, Aburano H, Ikenaga M, Nakashima M. Permissible residual deformation levels for building structures considering both safety and human elements. In: Paper presented at the Proceedings of the 14th world conference on earthquake engineering, Beijing, China; 2008.
- [53] Melo J, Fernandes C, Varum H, Rodrigues H, Costa A, Arêde A. Numerical modelling of the cyclic behaviour of RC elements built with plain reinforcing bars. *Eng Struct* 2011;33(2):273–86.
- [54] Mitchell D, DeVall RH, Saatcioglu M, Simpson R, Tinawi R, Tremblay R. Damage to concrete structures due to the 1994 Northridge earthquake. 1995;22(2):361–77. doi:10.1139/195-047.
- [55] Moehle JP. Displacement-based design of RC structures subjected to earthquakes. *Earthquake Spectra* 1992;8(3):403–28. <https://doi.org/10.1193/1.1585688>.
- [56] Moffatt K, Dowling PJ. Shear lag in steel box girder bridges. *Struct Eng* 1975;53 (10).
- [57] Muir-Wood R. The Christchurch earthquakes of 2010 and 2011. The Geneva risk reports. Risk and insurance research. Extreme events and insurance: 2011 Annus horribilis. Courbage C, Stahel WR, Geneva. In. 2015.
- [58] Nakashoji BA. Seismic performance of square nickel-titanium reinforced ECC columns with headed couplers. Reno: University of Nevada; 2014.
- [59] Paulay T. The design of ductile reinforced concrete structural walls for earthquake resistance. *Earthquake Spectra* 1986;2(4):783–823.
- [60] Paulay T. The Design of reinforced concrete ductile shear walls for earthquake resistance. University of Canterbury, Canterbury, New Zealand. (Research Report No. 81-1); 1986b.
- [61] Priestley MJN, Calvi GM, Kowalsky MJ. Displacement-based seismic design of structures / M. J. N. Priestley, Gian Michele Calvi, Mervyn J. Kowalsky; Pavia: IUSS Press : Fondazione Eucentre; 2007.
- [62] Priestley MJN, Park R. Strength and ductility of bridge substructures. In: *National roads board*, Wellington, New Zealand: RRU Bulletin 71; 1984.
- [63] Priestley MJN, Park R. Strength and ductility of concrete bridge columns under seismic loading. *Struct J* 1987;84(1). <https://doi.org/10.14359/2800>.
- [64] Roh H, Reinhorn AM. Hysteretic behavior of precast segmental bridge piers with superelastic shape memory alloy bars. *Eng Struct* 2010;32(10):3394–403.
- [65] Saiidi MS, O'Brien M, Sadrossadat-Zadeh M. Cyclic response of concrete bridge columns using superelastic nitinol and bendable concrete. *ACI Struct J* 2009;106 (1).
- [66] Saiidi MS, Wang H. Exploratory study of seismic response of concrete columns with shape memory alloys reinforcement. *ACI Mater J* 2006;103(3):436.
- [67] Sritharan S, Beyer K, Henry RS, Chai YH, Kowalsky M, Bull D. Understanding poor seismic performance of concrete walls and design implications. *Earthquake Spectra* 2014;30(1):307–34. <https://doi.org/10.1193/021713EQS036M>.
- [68] Standards Association NZ. (2006). NZS 3101: Part1:2006, Concrete Structures Standard - The Design of Concrete Structures.
- [69] Steinmetz M, Wyckmans T. Design of an experimental setup for cyclic testing of large-scale units at LEMSC. MSc. Thesis, UCLouvain [in French]; 2019. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2004\)130:4\(618\)](https://doi.org/10.1061/(ASCE)0733-9445(2004)130:4(618)).
- [70] Thomsen J, Wallace J. Displacement-based design of slender reinforced concrete structural walls—experimental verification. *J Struct Eng* 2004;130(4):618–30. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2004\)130:4\(618\)](https://doi.org/10.1061/(ASCE)0733-9445(2004)130:4(618)).
- [71] Tripathi M, Dhakal RP, Dashti F. Nonlinear cyclic behaviour of high-strength ductile RC walls: Experimental and numerical investigations. *Eng Struct* 2020;222: 111116.
- [72] Wallace JW, Moehle JP. Ductility and detailing requirements of bearing wall buildings. *J Struct Eng* 1992;118(6):1625–44.
- [73] Xue S, Li X. Control devices incorporated with shape memory alloy. *Earthquake Eng Vib* 2007;6(2):159–69.
- [74] Zuo X-B, Li A-Q, Chen Q-F. Design and analysis of a superelastic SMA damper. *J Intell Mater Syst Struct* 2008;19(6):631–9.