



Nonlinear macroelement for steel shear walls under seismic loading

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ABSTRACT

This paper proposes an efficient macroscale representation for unstiffened thin steel shear walls, enabling accurate and efficient predictions under earthquake loading. The 8-noded macroelement incorporates six nonlinear springs with asymmetric constitutive relationships to represent the cyclic response of steel panels. The macroelement formulation is introduced first, followed by the constitutive model for the nonlinear springs, which allows for strain-hardening in tension, strength degradation in compression, residual strength at the strain reversal point, and stiffness reduction. The material parameters for the constitutive model are calibrated by multi-objective optimisation with Genetic Algorithms based on the numerical results provided by accurate nonlinear FE models with shell elements. Linear regression is utilised to establish the material parameters for infill plates with different geometric properties. The residual of dissipated energy for the calibrated macroelement models lies between 3 % overestimation and 12 % underestimation compared to detailed shell element models, whereas the computational demand is reduced with wall-clock time reductions of more than 97 %. Finally, the proposed macroelement is verified in numerical examples, where a substandard RC frame enhanced with different types of steel shear walls is analysed under cyclic loading using detailed shell element models and the proposed macroelement for the steel wall components. The excellent comparisons confirm that the proposed macroelement model provides an efficient and accurate description of unstiffened steel wall components, and it can be used for realistic nonlinear dynamic simulations of framed buildings equipped with steel walls under earthquake loading.

1. Introduction

Steel shear walls can be effectively used to improve the seismic performance of substandard framed structures designed without consideration of earthquake loading [1,2]. In typical applications, steel thick plates, stiffened or unstiffened thin plates, are connected to existing frame components, forming strong and stiff infill panels. The solution with unstiffened thin steel panels, which is the most utilised in practice due to reduced costs and enhanced constructability [3], has been recently studied by experimental and numerical research [1,2,4–6]. Adding unstiffened thin steel plates can provide high lateral stiffness and shear strength to deficient frame buildings. Moreover, the supplementary steel plates remain elastic during minor seismic actions. Still, they absorb substantial energy by forming a post-buckling tension field mechanism under more significant earthquakes. Thus, the hysteretic energy dissipation capacity of the retrofitted frame structure can be significantly enhanced, increasing the seismic resilience at the same time [7]. Besides, using steel shear walls or new sustainable concrete, i.

e. geopolymer concrete [8,9], to substitute traditional Portland concrete shear walls can lower CO₂ emissions, leading to more energy-efficient and environmentally friendly retrofitting solutions. Previous research showed that the complex nonlinear response of steel shear walls can be accurately predicted using nonlinear finite element (FE) models with shell elements [3,10,11]. However, the use of detailed models with nonlinear shell elements allowing for material and geometric nonlinearity poses significant computational demands. Thus, a simpler and more efficient modelling strategy is required for representing steel shear walls within large building models when investigating the seismic performance of retrofitted structures.

Efficient simplified descriptions for steel shear walls were proposed in previous studies, where several strips or braces are employed to simulate the steel wall behaviour. These models aim to accurately predict the initial stiffness and ultimate strength under in-plane horizontal forces. Thorburn et al. [12] first proposed a tension strip model as shown in Fig. 1(a), where each panel is modelled as a series of tension-only strips with an inclination angle of the tensile field developed after the

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panel buckling. According to [13], the inclination angle is derived by minimising the internal work done by the plate allowing also for the contributions from the axial forces in the beam and columns connected to the steel wall component. Timler and Kulak [4] modified the derivation for the inclination angle by considering an additional effect of the bending strain energy term for the columns. Such an approach is adopted by the Canadian standard CAN/CSA S16-01 [13] and the American AISC seismic provisions [14]. Fig. 1(b) shows the equivalent truss model with a single pin-ended diagonal brace member proposed by Thorburn et al. [12], which is recommended for the preliminary seismic design of buildings with steel shear walls in CAN/CSA S16-01 [13]. Notably, the multi-strip model neglects the contribution from the compressive stresses in the orthogonal direction of the tension field to the strength and stiffness of the steel panel [15], while the equivalent truss model disregards the distributed forces of the web plate applying to the boundary frame and overestimates the ultimate strength of steel shear wall [3,16]. Based on the previous two models, Shishkin et al. [15] modified the multi-strip model by adding a compression truss with the same cross-sectional area as the equivalent truss model and damaged hinges, as illustrated in Fig. 1(c). Subsequently, Tian et al. [17] proposed a three-strip model based on the multi-strip formulation, as presented in Fig. 1(d). Three tension-only strips are employed in the diagonal direction and connect to the opposite corner and the mid-span of the boundary frame. This model is capable of capturing the storey stiffness and column reaction forces, but it may lead to inaccurate prediction of the ultimate strength of the steel panel.

It is important to note that most previous simplified models were developed to represent mainly the response under monotonic loading conditions, while only a few attempts have been made to describe the hysteresis behaviour under cyclic loading. Choi and Park [3] proposed a hysteretic model based on the stress-strain relationship obtained from shell element results and then applied it to the multi-strip model. Jalali and Banazadeh [18] modified the Choi-Park model by including cyclic and in-cycle deterioration while retaining the same pinching rule for unloading from compression and reloading in tension. Wang and Yang [19] defined skeleton curves and hysteretic criteria for the multi-strip model considering the cyclic hardening behaviour of four different steel materials and residual stress under compression. The hysteretic models developed thus far, however, enable accurate predictions of the cyclic response of steel shear wall panels with generic geometrical characteristics, including strength and stiffness degradation and energy

dissipation, only when assuming specific model material parameters [19], as a comprehensive calibration of existing hysteretic models material parameters has yet to be carried out due to the lack of experimental data for a suitable range of steel wall panel configurations.

In this paper, a new accurate and efficient modelling strategy for unstiffened thin steel shear walls is proposed and implemented into an 8-noded macroelement description, which can be conveniently connected to the beams and columns of a frame model. The macroelement incorporates six nonlinear springs with asymmetric constitutive relationships to represent the cyclic nonlinear response of steel shear panels. The material parameters for the constitutive model are calibrated based on the shell element results under cyclic loading. The non-dominated sorting genetic algorithm II (NSGA-II) [20] is employed to find optimal solutions, minimising the discrepancy between the results obtained using the developed macroelement and accurate shell element models for steel shear walls. Linear regression for the material parameters in terms of infill plate properties is also carried out, allowing for various plate lengths, heights and thicknesses. It provides a practical definition of constitutive macroelement model parameters for accurate and efficient nonlinear simulations of steel shear wall components. Finally, the calibrated macroelements are compared against experimental results for steel plate shear walls with different aspect ratios and detailed FE models with shell elements of a representative substandard RC frame equipped with different types of steel shear walls.

2. Macroelement representation

2.1. Macroelement formulation

The proposed 8-noded macroelement, which has been implemented

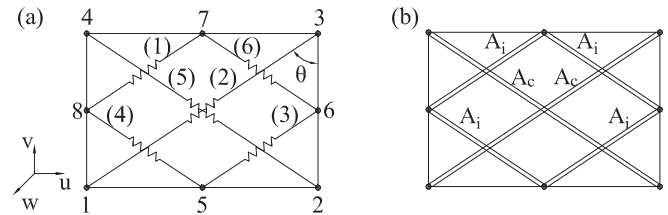


Fig. 2. (a) Steel shear wall macroelement; (b) Equivalent braces.

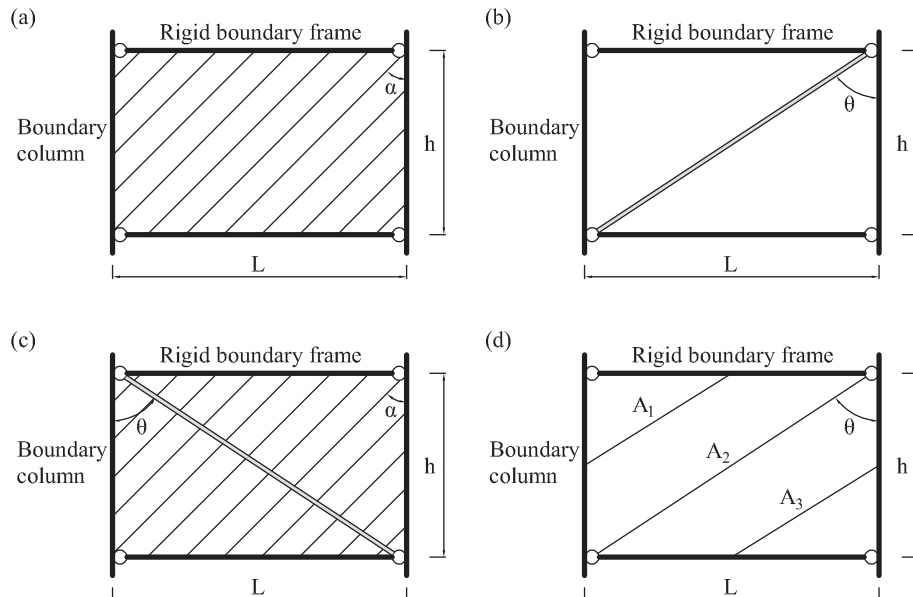


Fig. 1. Simplified models: (a) Multi-strip model [12]; (b) Equivalent truss model [12]; (c) Modified strip model [15] (d) Three-tension-strip model [17].

in the FE program ADAPTIC [21], includes four internal and two corner nonlinear springs, as sketched in Fig. 2(a). Each spring represents an equivalent component of the braced model depicted in Fig. 2(b). The tangent stiffness of each spring is determined as:

$$K_{c(i)} = \frac{S \cdot A_{c(i)}}{L_{c(i)}} \quad (1)$$

where S is the material tangent stiffness depending on the adopted constitutive relationship; $A_{c(i)}$ is the cross-section area of the equivalent corner (c) or internal (i) brace component; $L_c = \sqrt{L^2 + h^2}$ and $L_i = \sqrt{L^2 + h^2}/2$ represent the length of the corner and internal springs, respectively. The axial force for each spring is given by the stress defined by the material model multiplied by the cross-section area, denoted as:

$$F_{c(i)} = \sigma \cdot A_{c(i)} \quad (2)$$

The equivalent brace areas A_c and A_i are set according to the three-strip model proposed by Tian et al. [17], using the following expressions:

$$A_c = \frac{ht \sin^2 \alpha \cos^2 \theta}{\sin \theta \cos^2 \theta} - \frac{2ht \sin^3 \alpha \cos \alpha}{3 \sin^2 \theta \cos \theta} \quad (3)$$

$$A_i = \frac{2ht \sin^3 \alpha \cos \alpha}{3 \sin^2 \theta \cos \theta} \quad (4)$$

where L , h , t are the length, height and thickness of the infill steel wall; $\theta = \tan^{-1}(L/h)$ is the angle between the corner brace and the vertical direction; α is the inclination angle of the tension field, which is derived by minimising the internal work due to the tension field within the steel panel component, and it can be simplified as $\alpha = 45^\circ$ according to Wang and Yang [19].

The macroelement is developed according to a 3D FE framework. The eight nodes are arranged on the external edges of the steel wall panel. The order of the node connectivity is shown in Fig. 2(a). Each node is characterised by three translational degrees of freedom (DOFs) noted as u , v , w in the local element coordinate system. The element DOF vector (24×1) is therefore expressed as:

$$\{u\} = \{u_1, v_1, w_1, \dots, u_8, v_8, w_8\}^T \quad (5)$$

Six nonlinear springs connect different pairs of the corner and mid-edge nodes assuming a specific constitutive relationship allowing for degradation of strength and stiffness under cyclic loading, as discussed in the following section. The compatibility conditions of the macroelement are expressed as:

$$\{d\} = [T_r] \{u\} \quad (6)$$

where $\{d\}$ (6×1) is the deformation vector collecting the axial deformations of the springs; $[T_r]$ (6×24) is the geometric transformation matrix, which can be derived based on first-order kinematics under small displacements and the element connectivity.

2.2. Material constitutive model

The constitutive material model for the diagonal springs of the macroelement is developed to represent the typical response characteristics of steel shear walls under in-plane cyclic loading. This material model is derived from a complex interaction between plate buckling and steel yielding, including the strain-hardening after yielding, residual shear force after each loading cycle, elastic stiffness degradation under cyclic loading and pinching and sliding upon reloading.

The adopted constitutive model is based on the hysteretic uniaxial stress-strain law put forward by Choi and Park [3]. It is mainly governed by nine parameters $\{E_s, f_y, \mu, \beta, \gamma, \chi, \eta, \kappa, \rho\}$. More specifically, the elastic tangent stiffness is set as the Young's modulus E_s of structural steel. Steel yield strength f_y is multiplied by the factor η to obtain the elastic limit in

tension f_{tp} , and by the effective area factor ρ for Class 4 cross-sections [22,23] and the buckling coefficient χ to determine the compressive strength f_{cp} . Strain-hardening in tension is governed by the material parameter μ , and degradation in compression is defined by β . Additional factors γ and κ are introduced into the constitutive model to control the residual strength at the strain reversal point and the degree of stiffness reduction, respectively. Fig. 3(a) schematically shows the evolution of the stress-strain relationship starting from loading in tension. In the first loading cycle, a bilinear curve is followed with initial elastic stiffness E_s up to yielding at strain $\varepsilon_{t0} = f_{tp}/E_s$ (point TA) and reduced stiffness μE_s for larger deformations. When unloading after yielding, stresses reduce linearly from the stress at point TB $[\varepsilon_{TB}, f_{TB}]$ followed by a linear branch with stiffness equal to the elastic stiffness E_s . After reaching the compressive strength limit f_{cp} at TC, the compressive stress degrades with a slope of βE_s . When the strain increment changes sign from point TD, the stress increases linearly to the strain reversal point TE $[0, \gamma f_{TB}]$ where the second cycle starts. With increasing tensile strain, the elastic tensile stress grows until it reaches the plastic tensile curve at TF. The yielding tensile strain value in the second cycle ε_{TF} considers the degree of stiffness reduction κ and it is defined by $\varepsilon_{TF} = \varepsilon_{t0} + \kappa(\varepsilon_{TB} - \varepsilon_{t0})$. After the yield point, the stress-strain relationship follows the same rule as for the first loading cycle.

Fig. 3(b) displays the cyclic behaviour starting from loading in compression. Also, in this case, the initial elastic stiffness corresponds to E_s and a reduced softening stiffness βE_s is assumed after yielding in compression from point CA at $\varepsilon_{c0} = f_{cp}/E_s$. Unloading from CB follows a linear branch up to the strain reversal point CC $[0, \gamma f_{tp}]$. Then, the tensile yielding point CD $[\varepsilon_{t0}, f_{tp}]$ is reached followed by a linear branch with reduced stiffness μE_s .

3. Calibration and definition of the material parameters for the constitutive model

3.1. Calibration methodology

The key material parameters: strain-hardening in tension μ , degradation in compression β , residual strength factor at strain reversal factor γ , shear strength deviation χ , buckling reduction factor η and degree of stiffness degradation κ for the nonlinear springs of the a macroelement representing a specific steel wall panel are obtained by employing a multi-objective optimisation strategy, where nonlinear FE element

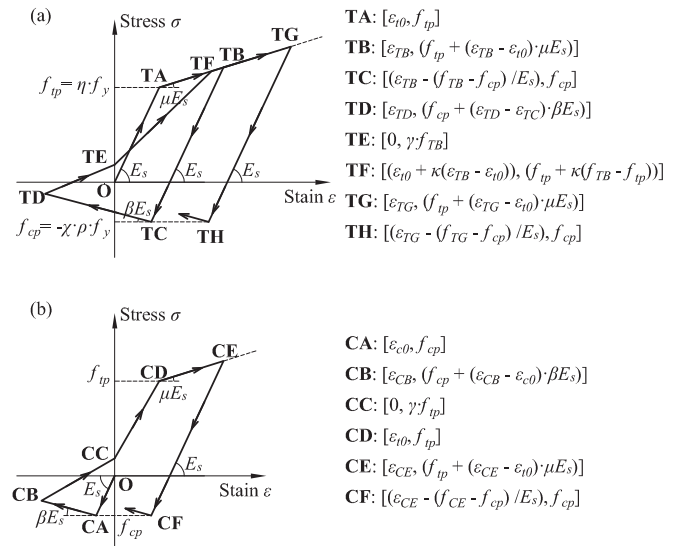


Fig. 3. Material constitutive model: (a) Initial loading in tension; (b) Initial loading in compression.

models with shell elements provide reference numerical results. Such numerical models are adopted because of the lack of experimental data on steel shear walls with different geometrical characteristics subjected to cyclic loading.

According to the proposed calibration strategy, the non-dominated sorting Genetic Algorithm (GA) NSGA-II [20] for multi-objective optimisation is applied to find the solutions to the optimisation problem for model calibration. Compared to other multi-objective genetic algorithms, such as the Pareto-archived evolution strategy and the strength-Pareto evolutionary algorithm, NSGA-II offers a significantly better distribution of optimal solutions and improved convergence towards the true Pareto front [20]. Thus, it has been selected as the optimisation method for this research. The optimal set of material parameters $\mathbf{p} = \{\mu, \beta, \gamma, \chi, \eta, \kappa\}$ for a given panel configuration is obtained by minimising objective functions representing discrepancies between some response characteristics predicted by the baseline FE model with shell elements and the macroelement counterpart.

The transition from the shell element to the macroelement first considers the dissipated energy equivalence for the steel shear wall component, which can be achieved by equating the work done by the total shear force for the analysed steel panel. Thus, according to the procedure presented in Chisari et al. [24] and Chisari et al. [25], the first objective function f_1 allows for the discrepancy in dissipated energy predicted by the macroelement and the associated shell element model. However, numerical tests found that minimising only the energy discrepancy may result in an inconsistent shape of the resulting macroelement force–displacement curve and difficulties in achieving convergence within an acceptable maximum number of iterations, which is mainly due to the relatively large number of material parameters to be calibrated by the optimisation process. Thus, additional response characteristics are introduced into the optimisation formulation by defining a second objective function f_2 which considers some engineering features Φ extracted from the shear force–displacement curves for both shell and macroelement models. The response characteristics include elastic stiffness, yielding force, maximum and residual forces after compression unloading. In this way, the multi-objective optimisation problem can be expressed as:

$$\begin{aligned} \text{Find } \mathbf{p} &= \{\mu, \beta, \gamma, \chi, \eta, \kappa\} \\ \min f_1(\mathbf{p}) &= \int_0^T [(F^M(\mathbf{p}t) - F^S(t))du/dt]^2 dt \\ \min f_2(\mathbf{p}) &= (\Phi^M(\mathbf{p}) - \Phi^S)^T \cdot \mathbf{W} \cdot (\Phi^M(\mathbf{p}) - \Phi^S) \end{aligned} \quad (7)$$

where $F^M(\mathbf{p}, t)$, $F^S(t)$ are the base shear forces of the macroelement and shell element model, respectively;

du/dt is the incremental lateral displacement during the time increment dt ;

$\Phi = \{K_{in}, K_{fin}, F_y, F_{max}, F_{res}\}^T$ is the response characteristics vector;

K_{in}, K_{fin} are elastic stiffness in the initial and final loading cycle, respectively;

F_y, F_{max}, F_{res} are yielding force, maximum force and residual force after compression unloading;

$$\mathbf{W} = \text{diag} \left[\left(\frac{F_{max}^S}{K_{in}^S} \right)^2, \left(\frac{F_{max}^S}{K_{in}^S} \right)^2, 1, 1, 1 \right] \text{ is the weight matrix}$$

considering the different physical units in Φ .

The MATLAB function ‘gamultiobj’ is applied, which is based upon the NSGA-II procedure [20] and is available in the *Global Optimisation Toolbox* [26]. Considering the definition of the sought material parameters, the searching bounds are given as $\mu \in [0, 0.1]$, $\beta \in [-0.1, 0]$, $\gamma \in [0, 1]$, $\chi \in [0, 1]$, $\eta \in [0, 2]$, $\kappa \in [0, 1]$. Population size, crossover ratio, and the maximum number of iterations are set as 200, 0.8 and 1200, respectively, which are the recommended values for an optimisation problem with six variables [26]. For the mutation of this optimisation problem with constraints, the adaptive

feasible function method is selected to generate mutations randomly while still satisfying the assumed parameters bounds and linear constraints.

3.2. Numerical examples

The experimental test on the steel panel SW2 carried out by Roberts and Ghomi [27] shown in Fig. 4(a) is first selected to showcase the application of the proposed calibration procedure for the steel wall macroelement material properties. The physical test was performed on an unstiffened thin steel panel applying cyclic loading along the diagonal direction of the panel. The $300 \times 300 \text{ mm}^2$ steel panel with 0.83 mm thickness was mounted on a boundary square frame linked at the four corners by pinned joints. The steel panel was bolted to the boundary frame using high-strength 8 mm bolts assuring rigid connectivity between the panel and the frame. In the test, the specimen was pinned to a strong floor at one corner, while at the opposite corner a force along the diagonal direction was applied quasi-statically with controlled target displacements of $\pm 1.6 \text{ mm}$, 2.0 mm , 2.4 mm and 2.8 mm .

Initial numerical simulations are performed using a nonlinear FE model with shell elements, as shown in Fig. 4(b). The members of the boundary frame of the SW2 specimen are modelled using the cubic elasto-plastic beam-column elements ‘cbp3’ available in ADAPTIC [21], while 9-noded co-rotational shell elements ‘cvs9’ [11] are employed to represent the steel panel. Initial out-of-plane geometrical imperfections are introduced as half-sinusoidal shapes in the two main directions of the panel, with a maximum imperfection of 1 mm at the central node, to capture geometric nonlinear effects due to the out-of-plane buckling of the panel, as observed in the experimental test. Pinned joints connect the beam elements at the four corners of the frame. To represent the bolted connection in the physical specimen, the nodes of the beam elements of the boundary frame are connected to the nodes of the shell elements at the perimeter of the panel via specially designed joint elements, allowing some shear deformation in the plane of the specimen. The in-plane shear stiffness parameters of these joint elements are defined considering the contribution of the bolt double shear stiffness, which can be estimated as [28,29]:

$$\begin{aligned} C &= \left(\frac{t_1 + t_2}{2d} \right)^a \times \frac{b}{n} \left(\frac{1}{t_1 E_1} + \frac{1}{nt_2 E_2} + \frac{1}{2t_1 E_3} + \frac{1}{2nt_2 E_3} \right) \\ K_s &= 1/C \end{aligned} \quad (8)$$

where t_1 and t_2 are the thickness of the plate and straps, respectively; E_1 , E_2 and E_3 are the Young’s moduli of the plate, straps and bolt, respectively; d is the bolt diameter; a, b, n are coefficients and are equal to $a = 2/3$, $b = 3$, $n = 2$ for metal double shear joint. The resulting shear stiffness is $K_s = 6.12 \times 10^4 \text{ N/mm}$. The local x- and y-direction shear stiffnesses of the joints are released and set as the calculated value, as shown in Fig. 4(b), while all the other stiffness parameters are set as rigid.

Fig. 4(c) compares experimental and numerical results, indicating a good agreement. The shell element model leads to an accurate prediction of the panel stiffness and shear capacity and the main features of the cyclic response, including the pinching behaviour and the degradation of stiffness by increasing the number of cycles. The main discrepancy is related to the over-prediction of the elastic stiffness at the end of unloading after yielding. This has also been found by other researchers [10] in the analysis of the same steel panel specimen using a nonlinear FE model with shell elements in ANSYS, as shown in Fig. 4(d), where the ADAPTIC model produced closer and better predictions in terms of stiffness and strength.

The optimisation strategy with two objective functions introduced before has been applied to calibrate the material parameters of the macroscale model for the analysed single steel shear wall under a shear deformation mode. The FE model is restrained at the base, and a horizontal in-plane cyclic displacement is applied at the top edge, consid-

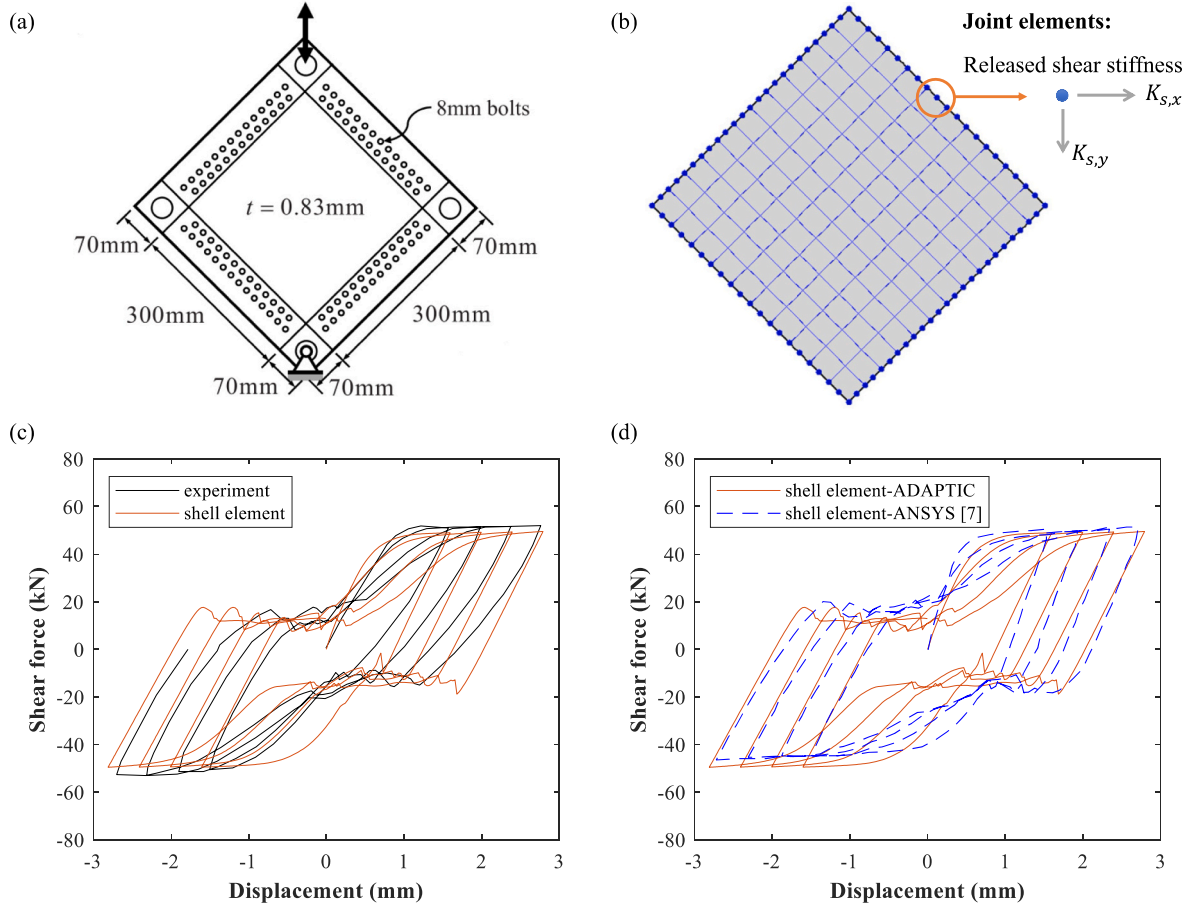


Fig. 4. (a) Experimental test of steel shear wall [25]; (b) FE model with shell elements; (c) Comparison between experimental and numerical results with shell elements in ADAPTIC; (d) Comparison between FE models with shell elements using ADAPTIC and ANSYS [10].

ering the same cyclic displacement history of SW2. For the ease of evaluating the errors of the two objective functions and selecting the optimal case, the objectives for each case on the Pareto Front are transformed to relative values as

$$\begin{cases} f_1(\mathbf{p}) = \frac{\int_0^T [(F^M(\mathbf{p}, t) - F^S(t)) du/dt]^2 dt}{\int_0^T (F^S(t) \cdot du/dt)^2 dt} \\ f_2(\mathbf{p}) = \frac{(\Phi^M(\mathbf{p}) - \Phi^S)^T \cdot \mathbf{W} \cdot (\Phi^M(\mathbf{p}) - \Phi^S)}{(\Phi^S)^T \cdot \mathbf{W} \cdot \Phi^S} \end{cases} \quad (9)$$

Fig. 5(a) shows Pareto Front for the solutions to the optimisation problem. Fig. 5(b) compares the numerical prediction by the shell element model against the numerical curves obtained by the macroelement model with the different sets of calibrated parameters associated with the different solutions of the multi-objective calibration. As can be seen from the figure, the relative errors of both objectives are negligible for all the cases on the Pareto Front, which leads to the very similar load-displacement macroscale curves in Fig. 5(b) all very close to the baseline shell element curve. This confirms that the adopted GA optimisation procedure returns suitable optimal results for the multi-objective optimisation problem. For a definitive selection of the sought material parameters, the case with the minimum summation of the 1st and 2nd relative objective values, which is marked with a black circle in Fig. 5(a), is chosen as the representative solution of the Pareto Front cases. Fig. 5(c) shows the numerical result comparison of the selected optimal case with material parameters $\mu = 0.0253, \beta = -0.0406, \gamma = 0.2165, \chi = 0.0459, \eta = 0.9676$ and $\kappa = 0.7760$, which provides a very good representation of the nonlinear response of the

analysed steel shear wall as predicted by the more detailed shell element model, including elastic stiffness, maximum shear force and stiffness degradation upon unloading and subsequent reloading.

For the SW2 specimen with the boundary frame, Fig. 5(d) and Table 1 compare the experimental and numerical results with shell elements and macroelements in terms of shear force-displacement curve under cyclic loading and selected response characteristics, respectively. It can be noted that both the shell element model and the macroelement model led to an accurate prediction of the actual shear strength, structural stiffness and hysteretic energy dissipation capacity.

3.3. Linear regression for material parameters

For the ease of applying the proposed macroelement description in nonlinear simulations of structures incorporating steel shear walls under earthquake loading, simple expressions have been determined to derive the material model parameters $\{\mu, \beta, \gamma, \chi, \eta, \kappa\}$ from the main geometrical characteristics (L, h, t) of steel wall elements using multiple linear regression (MLR). To achieve this, model calibration on a large population of realistic panel configurations with various lengths, heights and thicknesses has been carried out to find optimal sets of material parameters.

Virtual tests using shell elements are first performed to represent the response of steel shear walls under a shear deformation mode, because of the impracticality of using data from experimental tests. Then the calibration procedure is carried out based on the results from shell element models.

For the selection of numerical samples for the virtual tests, the steel wall height L is set as 500 mm to 6000 mm with a 500 interval.

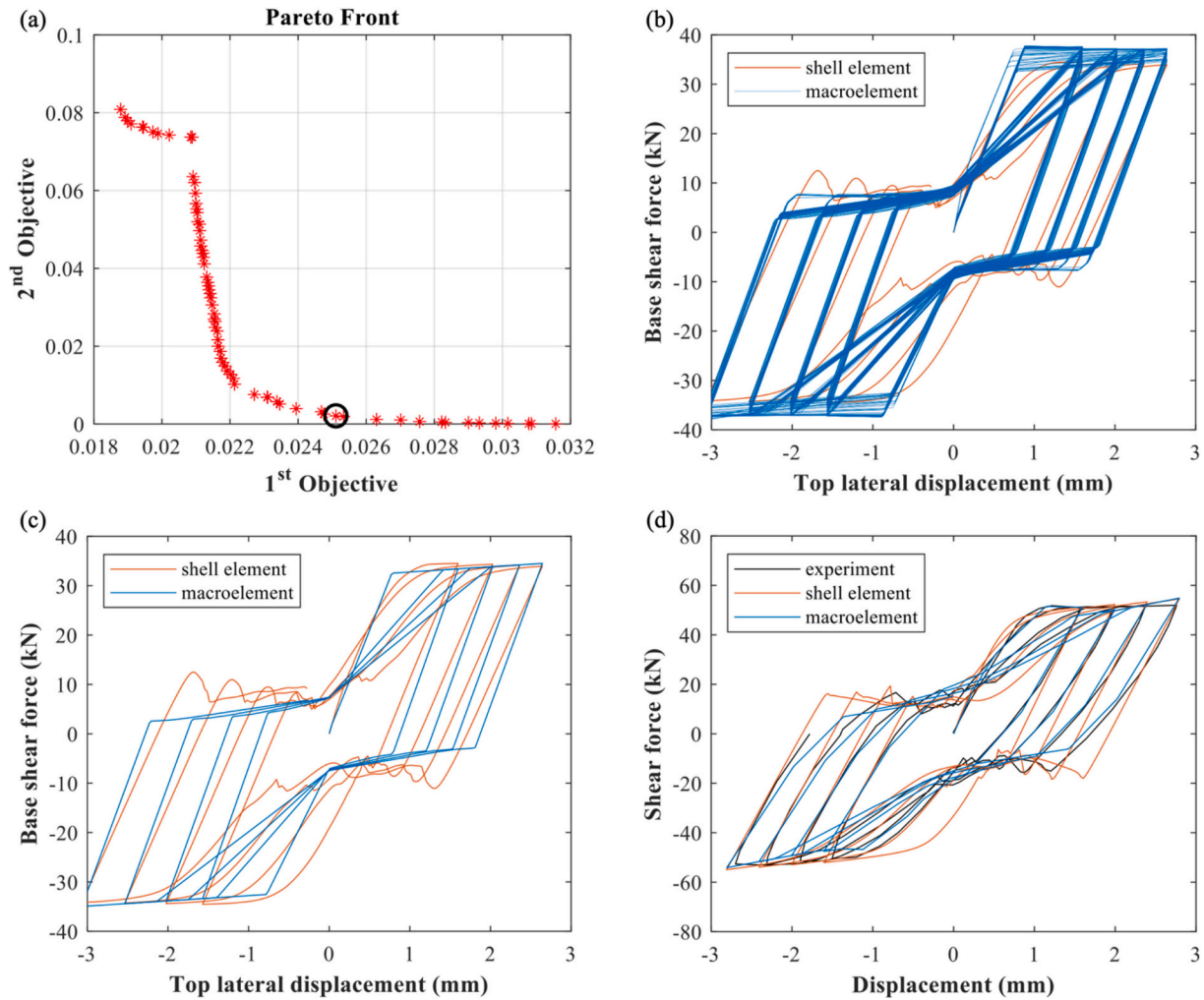


Fig. 5. (a) The optimisation Pareto Front for $p = \{\mu, \beta, \gamma, \chi, \eta, \kappa\}$; (b) Shell model load-displacement curve compared against macroelement curves with optimal solutions; (c) Shell model load-displacement curve compared to the macroelement curve with the selected optimal solution for the steel panel of the SW2 specimen; (d) Comparison among experimental, shell element and macroelement results for the SW2 specimen with the boundary frame.

Table 1

Selected response characteristics from the experimental test on panel SW2 and predicted by shell element and macroelement models.

	Dissipated Energy (kNmm)	Loading stiffness for the initial cycle K_{in} (kN/mm)	Loading stiffness for the last cycle K_{fin} (kN/mm)	Yield strength F_y (kN)	Ultimate strength F_{max} (kN)	Residual strength F_{res} (kN)
Experiment	573.8	61.9	20.3	51.8	52.0	17.4
Shell element	605.2	76.8	23.5	47.1	54.5	15.8
Macroelement	534.2	73.8	18.6	51.5	54.8	16.6

Considering the general design requirements and drift limits of boundary elements referred to ANSI/AISC 341–16 [14], the aspect ratios of length over height and length over thickness are typically limited to $0.8 \leq L/h \leq 2.5$ and $250 \leq L/t \leq 1000$, respectively. Therefore, for each length L , the height h is generated between $L/2.5$ and $L/0.8$ with 20 evenly spaced values rounded to centimetre and no larger than 4000 mm in view of realistic storey heights. Up to 10 equal-difference values of thickness t are selected within the range $L/1000$ and $L/250$ and rounded to integer numbers. A mesh of 12×10 9-noded shell elements [11] is adopted to model each sample, where Young's modulus and yield

strength of the steel material are equal to 210,000 MPa and 235 MPa, respectively. The shell element model is fully restrained at the bottom edge. Considering the rehabilitation drift requirements for RC framed structures under collapse performance level [30] and experimental test records [31], the maximum lateral drift is limited to 6.0% of the steel panel height, which corresponds to the inter-storey height of the retrofitted frame building. The shear force – top displacement cyclic curves for each sample are obtained by performing nonlinear simulations using ADAPTIC [21]. The dataset obtained from the macroelement calibration procedure is then randomly separated into 85 % for training and 15 %

Table 2
RMSEs for material parameters.

	μ	β	γ	χ	η	κ
Training data RMSE	0.0033	0.0106	0.0560	0.0347	0.0562	0.0924
Testing data RMSE	0.0033	0.0109	0.0578	0.0389	0.0590	0.0947

for testing. For the training set, the linear regression model ‘fitlm’ available in MATLAB *Statistic and Machine Learning Toolbox* [26] is first applied to find initial relationships with only linear and first-order terms of (L, h, t) . The function ‘step’ is then used to automatically adjust the regression models to include up to cubic terms, so that only highly correlated 2nd- and 3rd-order predictors are added to the functions. Afterwards, the test data is substituted into the regression models to obtain predicted values which are compared with calibrated values, aiming to examine the fitness of the regression model. Root mean square errors (RMSEs) for each material parameter of both the training and testing dataset are listed in Table 2. RMSEs of testing data are slightly larger than those of training data, and all the values are relatively small compared with the parameter value range, indicating that the regression models can provide a good fitting but not overfitting for the analysed data. Fig. 6 and Fig. 7 indicate the linear regression results (surfaces and lines) in comparison with the calibration results (blue dots) of the samples with 3000 mm length and 3000 mm height 10 mm thickness, respectively. Both figures prove that all the regression models are capable of capturing the variation trends of the material parameters.

Energy dissipation capacity is one of the most critical characteristics of steel shear walls under cyclic loading. As a result, the dissipated energy, which is the area enclosed by the base shear force – lateral displacement curve, is selected to measure the residuals of the regression models. The difference in dissipated energy is calculated for each case for the shell element model (indicated by subscript *s*) and the macroelement model using calibrated (*mc*) or predicted (*mp*) material parameters, respectively. Fig. 8(c) presents the total residuals of dissipated energy for the predicted models of all the samples in comparison with the original data from the shell element model, which are induced by the errors stemming from both the calibration procedure shown in Fig. 8(a) and the regression estimation illustrated in Fig. 8(b). The total residual

of most samples is between a 3 % overestimation and a 12 % underestimation, indicating that the regression models for material parameters are quite satisfying and can capture the energy dissipation level of steel shear walls. The larger discrepancy in the underestimation of the dissipated energy is largely due to the piecewise linear characteristics of the material model used for the developed macroelement. In comparison, shell element models predict more convex load-displacement responses, as illustrated in Figs. 5(c) and 5(d). This leads the macroelement to provide an accurate and mostly conservative estimate of the dissipated hysteretic energy.

4. Numerical results

4.1. Validation of calibrated macroelement model

Further validation for the proposed macroelement model and the procedure for the calibration of key model material parameters has been carried out considering the experimental results obtained by Choi and Park [32]. The analysed specimens FSPW1, FSPW2 and FSPW3 shown in Fig. 9 corresponds to a three-storey steel frame equipped with steel shear panels with different aspect ratios fully welded to the frame members with various column section sizes. The beam-to-column joints of the frame are full strength rigid joints. In the physical test, horizontal in-plane cyclic forces were applied at the roof beam restraining the out-of-plane displacements at the top of the columns. The test was conducted under displacement control following the top displacement history: $\pm 0.2\delta_y, 0.4\delta_y, 0.6\delta_y, 0.8\delta_y, 1.0\delta_y, 1.5\delta_y, 2\delta_y, 3\delta_y, 4\delta_y, 6\delta_y, 8\delta_y, 10\delta_y, 12\delta_y$, where $\delta_y = 15\text{mm}$ corresponds to the yield displacement from a numerical pushover analysis under monotonic loading conducted using the FE code ABAQUS [32].

As for the previous model validation example, an initial numerical simulation has been carried out employing nonlinear shell elements for the steel panel components and elasto-plastic beam-column elements for the members of the steel frame. Fig. 10 display comparisons between the experimental response and the numerical prediction in terms of top horizontal displacement against the base shear. The shell-element curves are very close to the experimental results, confirming the accuracy of the FE model with shell elements in ADAPTIC. In this case also the actual initial stiffness and the stiffness upon unloading are properly

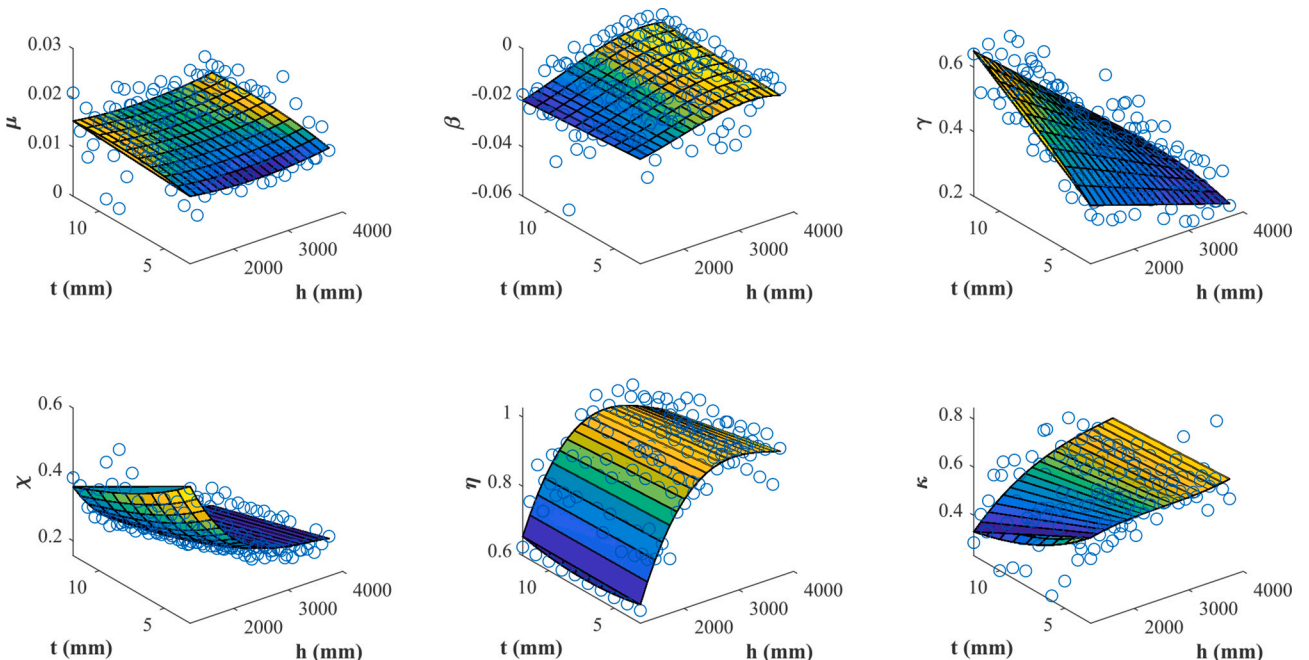


Fig. 6. Regression results for $L = 3000$ mm.

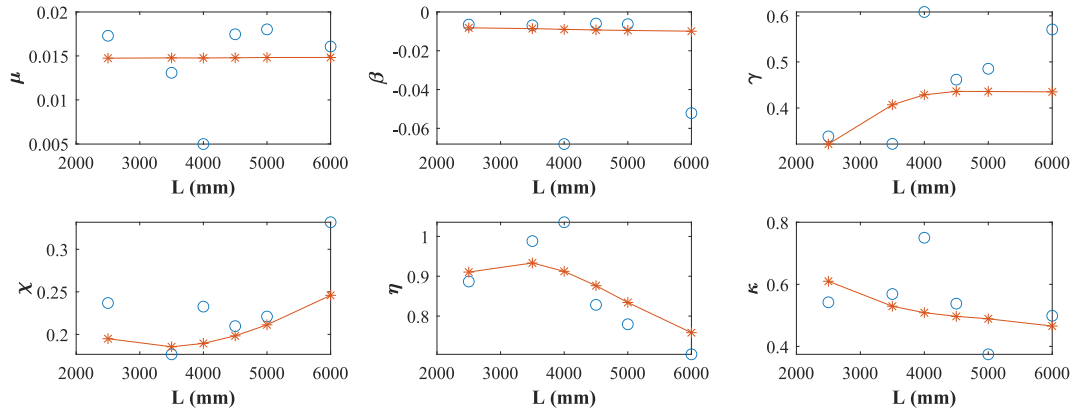


Fig. 7. Regression results for $h = 3000$ mm, $t = 10$ mm.

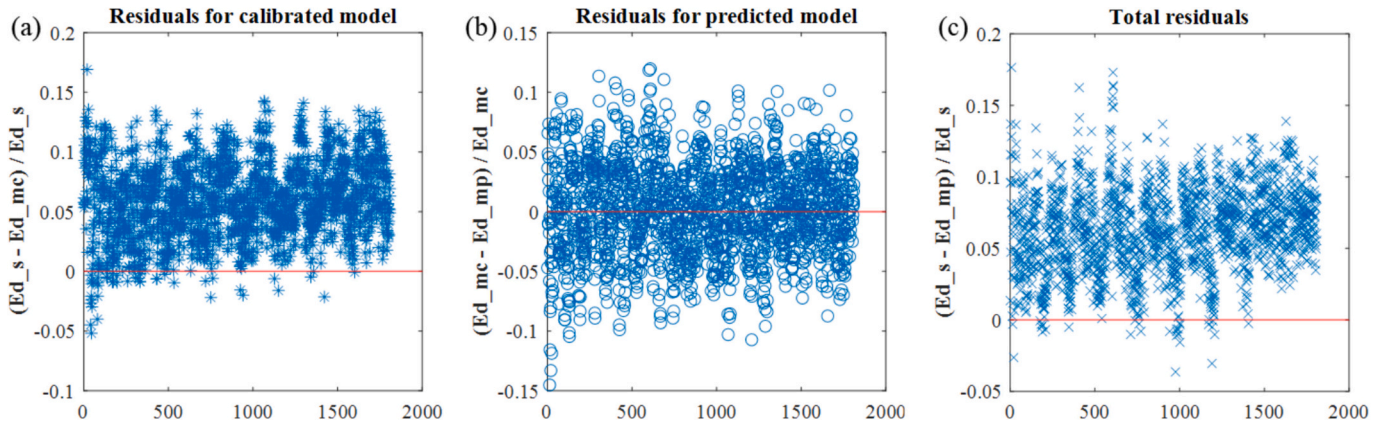


Fig. 8. Residuals of dissipated energy: (a) Calibrated model against the shell element model; (b) Predicted model against the calibrated model; (c) Total residuals for the predicted model in comparison with the shell element model.

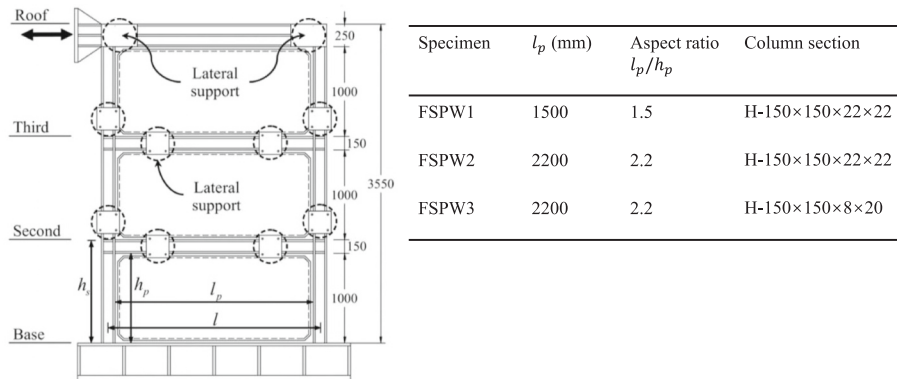


Fig. 9. Experimental 3-storey steel frame specimens with steel shear walls [32].

captured, possibly as the fully welded connection between the steel plates and the frame components are appropriately represented by the rigid connection between the shell elements of the steel plate model and the adjacent beam-column elements of the steel frame model. It can also be observed that the FE model with shell elements in ADAPTIC provides a more accurate prediction than the pushover curve obtained in [32] using ABAQUS.

In the subsequent stage of this numerical example, the calibrated macroelement material parameters for three numerical examples are defined using the MLR functions as described in the previous section. The comparison between the calibrated macroelement curve and the

numerical curve from the first stage of the validation for the FE model with nonlinear shell elements is shown in Fig. 10. The numerical curves for the macroelement models with different aspect ratios and frame section sizes are all in very good agreement with the shell element results, confirming further that the proposed modelling strategy with calibrated macroelements can be used for accurate predictions of frame structures with steel shear walls subjected to in-plane cyclic loading.

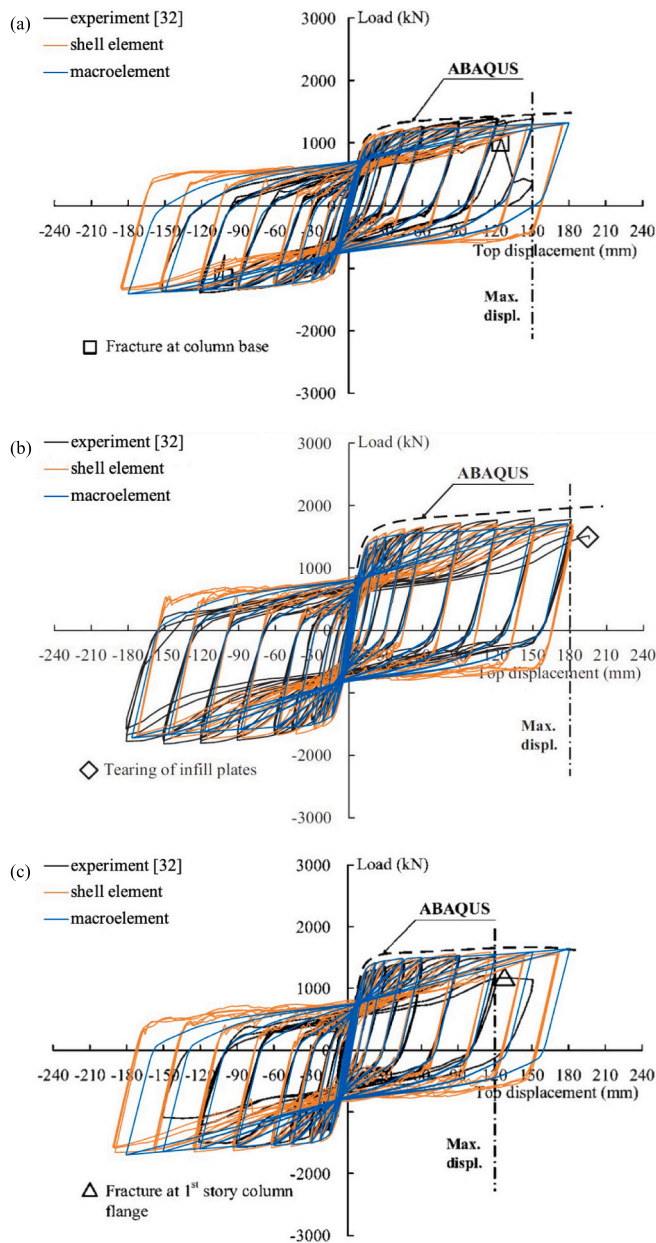


Fig. 10. Comparison between experimental [32] and numerical results with shell elements and calibrated macroelements for the steel panels of the specimens: (a) FSPW1; (b) FSPW2; (c) FSPW3.

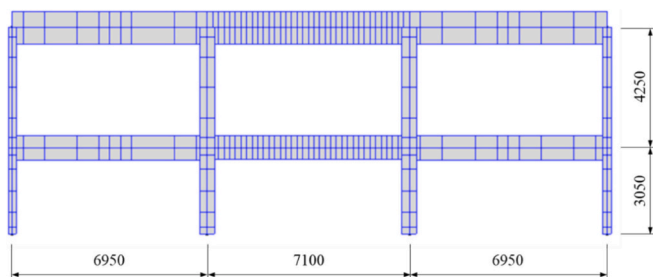


Fig. 11. FE mesh of the 2-storey RC frame (dimensions in mm).

4.2. Nonlinear simulation of the seismic response of a RC frame with steel shear walls

To assess the accuracy and efficiency of the proposed modelling strategy for steel shear walls further, the macroelement model has been employed to represent steel shear wall panels within a retrofitted 2-storey RC frame. The numerical results under cyclic loading have been compared against the numerical predictions obtained by representing the steel shear wall elements by an FE mesh with nonlinear shell elements.

The analysed 2D RC frame has been extracted from a realistic sub-standard 4-storey RC framed building built in Italy in the 1960s and investigated numerically under seismic loading in previous research [33]. The dimensions of the frame are shown in Fig. 11, where the FE mesh for the bare frame with nonlinear beam-column elements used for the nonlinear simulations is also displayed. The geometrical characteristics, material parameters and reinforcement details of columns and beams can be found in Appendix [34].

The adopted material model for steel reinforcement corresponds to the bilinear model 'stl1' in ADAPTIC (Fig. 12 (a)), while the elastoplastic behaviour of concrete is represented by the concrete model 'con1' (Fig. 12 (b)) which is characterised by softening branches in tension and compression to allow for concrete cracking and crushing. Table 3 reports the material properties for the RC frame members and the material model parameters utilised in the numerical simulations.

Steel shear walls with fully-infilled or partially-infilled plates have been considered in this numerical example. In both cases, the plates are 10 mm thick and made of structural steel with 235 MPa yield strength. The length of the fully-infilled walls corresponds to the bay length equal to 7100 mm, while the partially-filled wall is 2367 mm long (1/3 of the span). The two retrofitted frames are modelled using either one macroelement of a mesh of shell elements for each steel shear wall component at two floor levels. Fig. 13(a) and (b) show frame models with macroelements to represent fully-infilled and partially-infilled steel shear walls, while (c) and (d) displays retrofitted frame models where the steel shear walls are described by a mesh of shell elements.

The material parameters for the nonlinear springs of the macroelements, which are reported in Table 4, have been determined using the MLR functions established in the previous section based on the geometrical characteristics of the two steel shear wall components. Furthermore, to avoid introducing unrealistically high concentrated forces to the members of the RC frame, which are directly connected to the steel shear wall elements, only the four corner nodes of the macroelement are directly linked to the boundary frame, while the middle nodes on the four edges are connected to the corner nodes via link elements, which are pinned to the top and bottom nodes of the two boundary columns and have elastic axial stiffness and rigid twisting stiffness. The elastic axial stiffness allows for the stiffness contribution of the simulated steel panel, and it is assumed as the axial stiffness of a bar element with an area equal to one-quarter of the cross-section area of the steel plate. A rigid twisting stiffness is introduced to restrain the rotational DOF at the mid-node of the macroelement, which is not directly connected to a beam-column element; this restraint is introduced to avoid a nodal mechanism and has no impact on the modelling of the planar wall response.

Conversely, in the model with shell elements for the steel shear wall components, the shell elements are connected directly to the nodes of the boundary beams along the members in the horizontal direction, while the same link elements as the macroelement models are applied to the boundary nodes in the vertical direction.

4.3. Numerical results

Nonlinear simulations have been carried out employing the two alternative descriptions with macroelements and shell elements. The cyclic horizontal displacement history shown in Fig. 14 has been applied

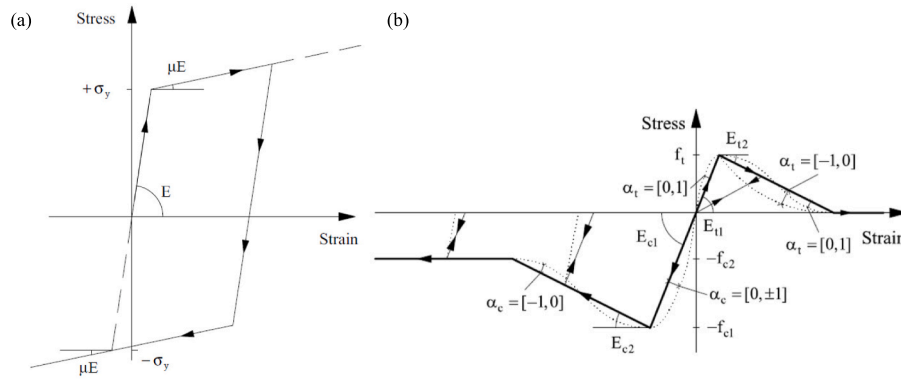


Fig. 12. (a) Steel reinforcement model 'stl1'; (b) Concrete material model 'con1' [35].

Table 3

Material properties for concrete and steel reinforcement (MPa).

Concrete compressive strength, f_{c1} (MPa)	Concrete residual compressive strength, f_{c2} (MPa)	Concrete tensile strength, f_t (MPa)	Concrete secant stiffness E_{c1}, E_{t1} (MPa)	Concrete softening stiffness E_{c2}, E_{t2} (MPa)	Steel reinforcement yield strength, f_y (MPa)	Steel Young's modulus, E_s (MPa)
18.7	18.7	0.001	26,544	0	382.5	210,000

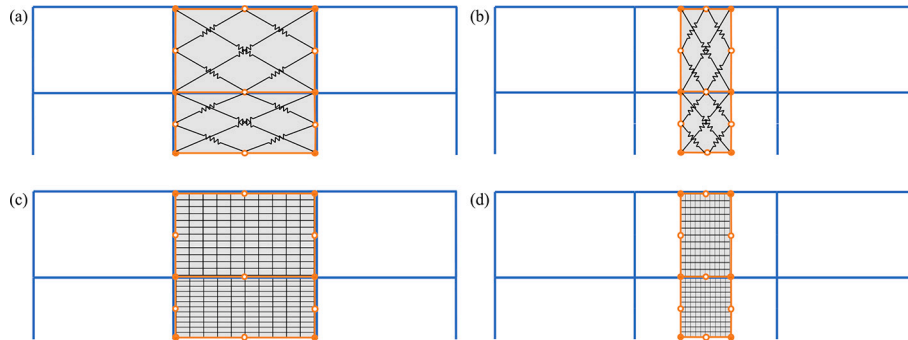


Fig. 13. Frame models with (a) macroelements for fully-infilled steel shear wall. (b) Macroelements for partially-infilled steel shear wall. (c) Shell elements for fully-infilled steel shear wall. (d) Shell elements for partially-infilled steel shear wall.

at the top beam restraining the out-of-plane displacements at the nodes where beams and columns are connected at two floor levels.

Fig. 15 compares the base shear-top displacement curves obtained using the two alternative models. It can be seen that using macroelement models for both the fully infilled and partially infilled cases leads to numerical predictions close to the baseline curves of the shell element models, with excellent estimates of the initial stiffness and yielding force. However, the level of stiffness degradation by increasing the number of cycles is underestimated, and the post-yielding behaviour is characterised by higher forces in the case of the fully-infilled frame. This may result from the different connectivity between the steel shear wall and the RC frame components in the shell element and macroelement models. On the other hand, the macroelement model guarantees a good estimation of the energy dissipated by the retrofitted frame under cyclic loading, with an overprediction of 0.9 % for the fully-infilled case and an underprediction of 1.2 % for the partially-infilled frame.

Table 4

Calibrated material parameters for nonlinear springs of macroelements in fully-infilled and partially-infilled models.

Model	Storey ID	μ	β	γ	χ	η	κ
Fully-infilled	1	0.0138	-0.0088	0.3741	0.2703	0.7325	0.4267
	2	0.0128	-0.0020	0.4044	0.2668	0.8007	0.4608
Partially-infilled	1	0.0157	-0.0087	0.2656	0.2002	0.8391	0.6425
	2	0.0173	-0.0119	0.1591	0.1565	0.7324	0.6766

Table 5 compares the computational costs for the numerical models in terms of wall-clock time and allocated memory. The analysis time for the macroelement model is reduced by over 97 % compared to the shell element model, and the memory usage is decreased by 83 %. This indicates that the computational demands are significantly lower when simulating steel shear walls using macroelements instead of shell elements.

Finally, it is worth comparing the distribution of internal forces due to the interaction between the frame components and the steel shear wall. Fig. 16 shows contour plots of the bending moment distribution in the beams and columns for the frame models with shell elements and macroelements at the maximum horizontal displacement. The macroelement models exhibit lower bending moments along the beams and higher bending moments at the column ends in the members directly connected to the steel shear walls. On the other hand, similar internal forces are predicted in the other beam-column elements of the frame,

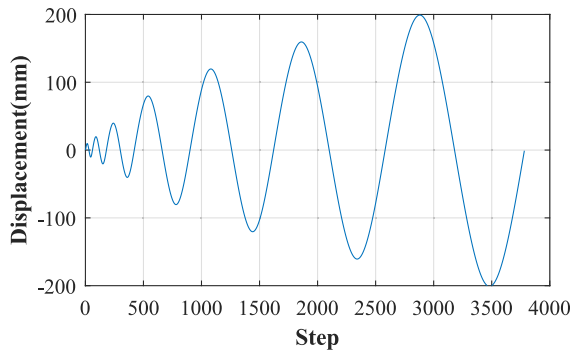


Fig. 14. Cyclic displacement history applied at the RC frame with a steel shear wall.

which are not directly connected to the steel wall. A difference of less than 15 % is observed when comparing the shear forces and the chord rotations for the frame members of both cases between the macroelement and shell element models.

This indicates that the use of the proposed macroelement strategy is suitable for the seismic assessment of retrofitted RC frame buildings when local checks for brittle failure modes (e.g. shear forces in beams and columns) and ductile failure modes (e.g. chord rotations) are limited to the elements not directly connected to steel shear walls. It is deemed adequate as beams and columns connected to steel wall components are generally reinforced to allow for the connectivity to the steel panels. On the other hand, it can be argued that local effects due to the interaction between steel walls and connected RC frame components can be investigated only using detailed models, such as the FE model with nonlinear shell elements.

5. Conclusions

In this paper, a novel simplified 8-noded macroelement formulation for unstiffened thin steel shear walls is proposed, which consists of six nonlinear springs with a newly defined asymmetric constitutive model. The model is capable of representing the cyclic nonlinear response of steel panels, including strain-hardening, residual shear strength with increasing load cycles, elastic stiffness degradation and pinching effects. Furthermore, the macroelement formulation benefits from a greatly enhanced computational efficiency, making it suitable for modelling large-scale structures under seismic loading.

The phenomenological formulation of the macroelement is discussed in detail, including the development of a tension field within the steel

panel and the contribution of the compressed parts of the panel when it is subjected to shear loading. The nonlinear constitutive model proposed for the nonlinear springs of the macroelement is also described, and the different model material parameters are specified. Besides, A calibration procedure is proposed for the material parameters of the developed macroelement, which is aimed at finding optimal sets of material parameters leading to response predictions close to those provided by detailed FE models with shell elements. The calibration is treated as a multi-objective optimisation problem considering the discrepancies of dissipated energy and selected engineering features between the macroelement and shell element results. The calibration procedure is applied to numerous samples of steel infill plates with various lengths, heights and thicknesses under a shear deformation mode. Subsequently, the calibration results are processed by MLR to determine simple functions for the practical calculation of macroelement material parameters in terms of the steel plate geometric properties.

The accuracy of the calibration results and the MLR functions have been assessed in the experimental results of steel plate shear wall specimens with different aspect ratios in comparison with shell element and macroelement numerical models and in the analysis of a sub-standard RC frame equipped with fully-infilled and partially-infilled steel shear walls modelled with macroelements or shell elements. It has been found that the results provided by the macroelement models are in good agreement with those of the shell element models, confirming the ability and computational efficiency of the developed macroelement in representing steel shear walls within retrofitted RC frames subjected to cyclic loading.

Future research will enhance the macroelement formulation by incorporating the effects of out-of-plane loading and the residual stresses that arise from the welded connections along the perimeter of the steel panel [36]. Additionally, efforts will focus on improving the connectivity between the macroelement and the connected frame components. This aims to create a more realistic distribution of internal forces within the beam-column elements directly interacting with the macroelements and to account for possible cracking or separation of the welded connections between the steel plate and the boundary elements [37].

Table 5

Computational costs for the numerical models with different elements.

Model	Analysis time	Allocated memory
Fully-infilled shell element	451 min	85 MB
Fully-infilled macroelement	3 min	7 MB
Partially-infilled shell element	131 min	35 MB
Partially -infilled macroelement	3 min	6 MB

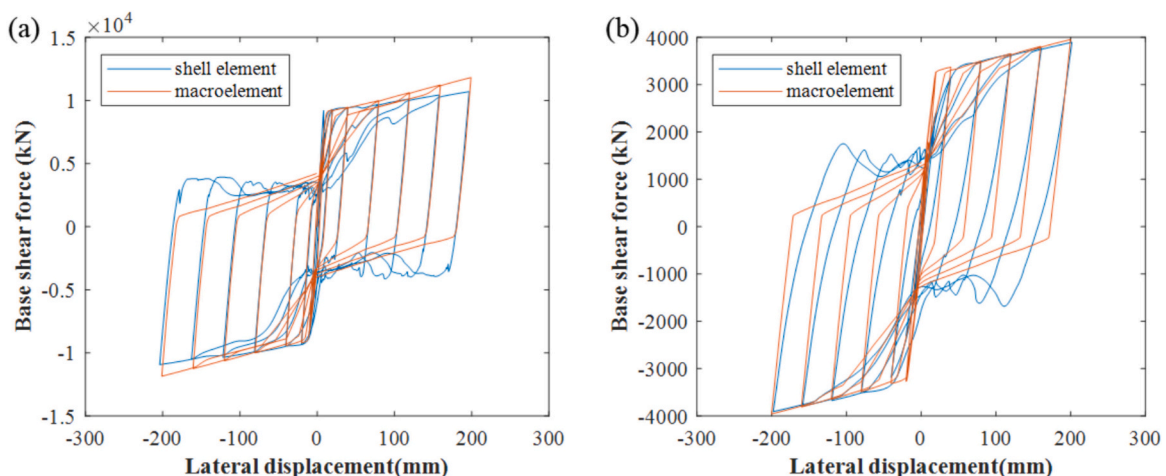


Fig. 15. Comparison between shell element and macroelement results for (a) fully-infilled model; (b) partially-infilled model.

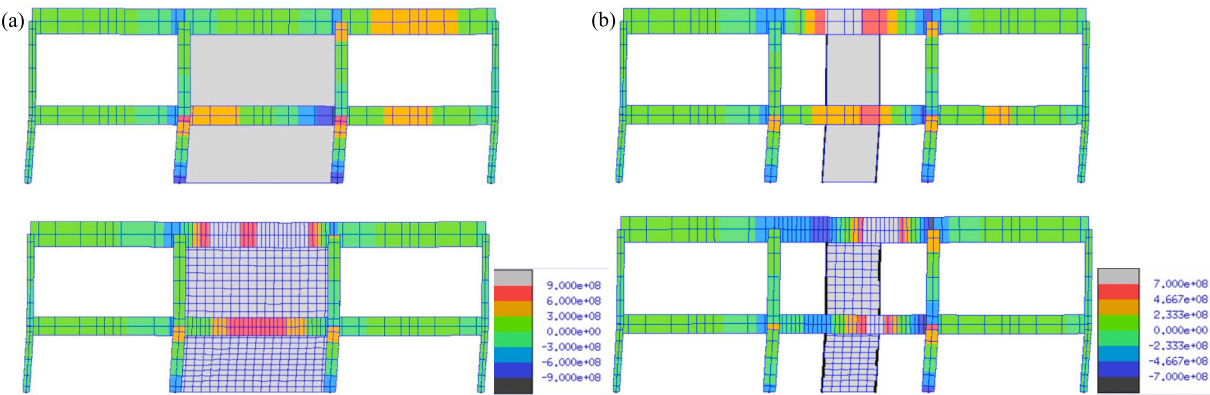


Fig. 16. Comparisons of bending moment distributions in the frame components: (a) Fully-infilled macroelement and shell element models; (b) Partially-infilled macroelement and shell element models.

CRediT authorship contribution statement

Xi Chen: Writing – review & editing, Writing – original draft, Validation, Methodology, Investigation, Formal analysis, Conceptualization. **Lorenzo Macorini:** Writing – review & editing, Supervision, Methodology, Conceptualization. **Bassam A. Izzuddin:** Writing – review & editing, Supervision, Software, Methodology, Conceptualization.

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.

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Appendix

Table 6
Geometrical characteristics of RC members of the 2-storey frame.

Member ID		Width (mm)	Depth (mm)
Columns	C31	650	300
	C32	550	550
	C33	550	550
	C34	650	300
Beams	B3231	300	850
	B3332	300	850
	B3433	300	850

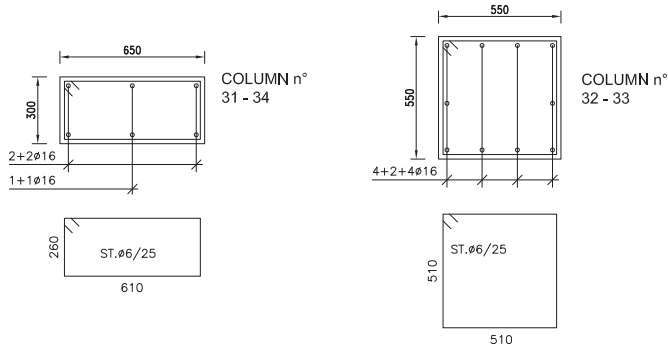


Fig. 17. Reinforcement details of columns on the 1st and 2nd floor (dimensions in mm) [34].

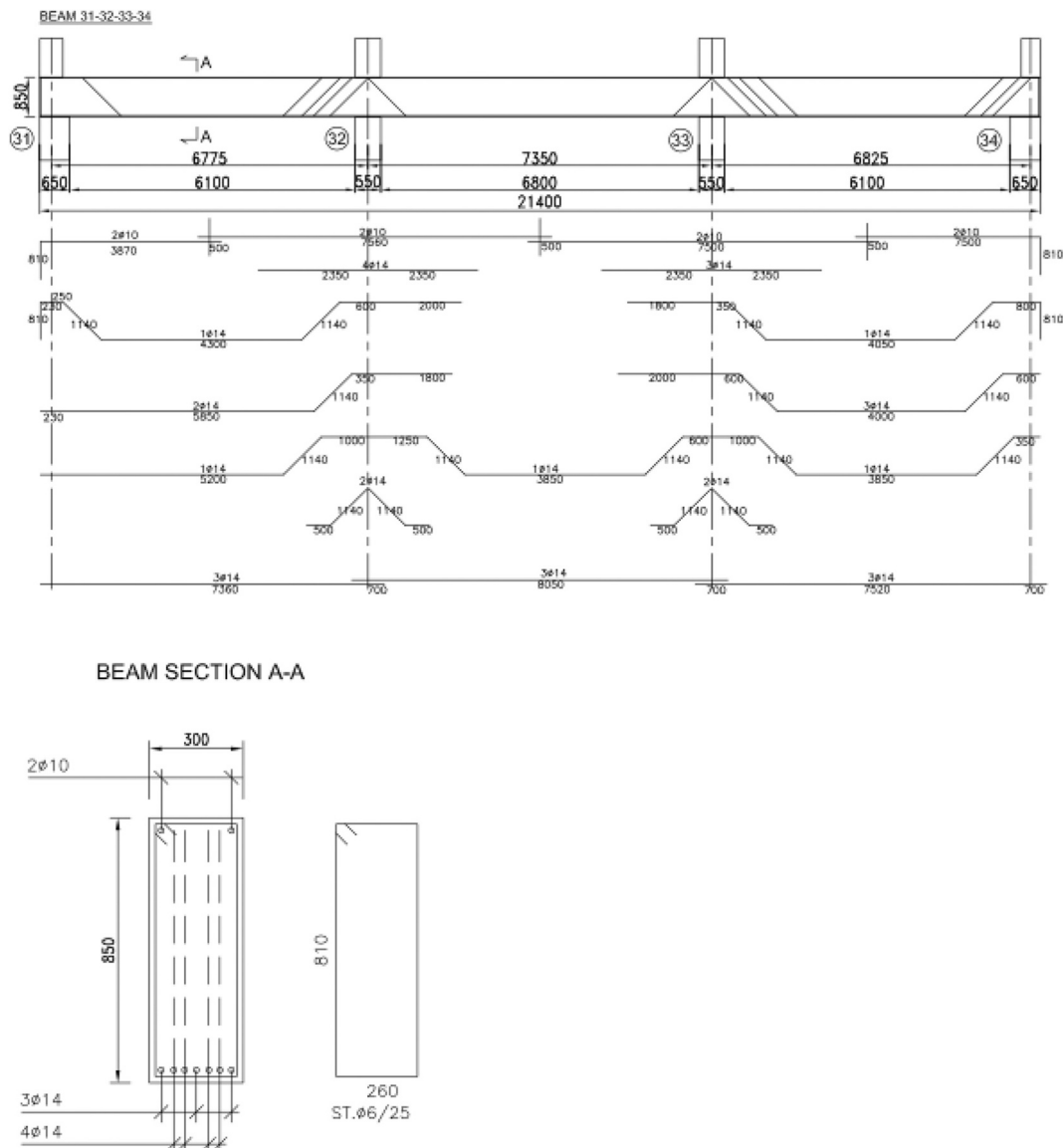


Fig. 18. Reinforcement details of Beam 31–32–33–34 at the 1st and 2nd floor (dimensions in mm) [34].

Data availability

Data will be made available on request.

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