STRUT-AND-TIE MODEL FOR A REINFORCED CONCRETE WALL STRENGTHENED WITH CARBON FIBER-REINFORCED POLYMERS

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Abstract
This study describes a nonlinear truss modeling approach for a reinforced concrete (RC) wall strengthened with carbon fiber-reinforced polymers (CFRP) subjected to a lateral displacement load test. Strengthening of a shear wall, constructed to replicate 1960 construction style, is done by bonding CFRP fabric bands on the wall two surfaces without any jacketing. The truss model is implemented using the commercially available software Autodesk Robot Structural Analysis. The capabilities of the model adopted are demonstrated by comparing the measured and computed load response behavior of non-strengthened RC walls and RC walls strengthened externally with CFRP. The proposed model depicted well both the shear strength and load response behavior of RC wall and RC wall strengthened with CFRP..

1. Introduction

It is well-known that the behavior of shear walls cannot be accurately described using conventional beam theory because of the interaction of flexure and shear. As a result, the analysis of shear walls has been a contentious issue for both researchers and structural engineers for decades. Researchers used the truss model to evaluate the linear and nonlinear behavior of RC structural elements subjected to monotonic and cyclic loading [1, 2]. Truss model to predict the shear capacity of an RC wall [3]. This model consisted of two vertical boundary elements to carry wall moments, diagonal compression members called struts that represent concrete and horizontal tie members representing shear steel reinforcement. The model appropriately predicts the shear capacity of a wall. Strut-and-tie model used to evaluate the shear strength of a squat wall for diagonal compression failure [4]. These two models approximated well the shear capacity of RC wall but were not capable to predict the wall load response behavior. Therefore, a new approach to truss model should be proposed that not only approximate the short wall shear strength but also depicts its load response behavior.

This study investigates the capability of the proposed truss model, by comparing the measured and computed load response behavior of non-strengthened RC walls and RC walls strengthened externally with CFRP. The walls used as control specimen have a number of design and detailing deficiencies, including the presence of lap splices of the longitudinal reinforcement at the base of the wall, poor confinement of the boundary elements, and poor anchorage of the transverse reinforcement. These details were representative of very poorly detailed short walls designed before the development of modern seismic design codes and were similar to the details used in the experimental program carried out by Greifenhagen and Lestuzzi [5] to assess the seismic performance of short walls.

2. Experimental detail

In this study, the control specimen represents at 1:3 scale the lower part of the shear wall of a building constructed in late 1960 in Switzerland prior to the introduction of earthquake-resistant design recommendations into building codes. The effective height of the walls was 610 mm, the width was 900 mm and the thickness was 80 mm. The properties of material used: concrete compressive strength 30 MPa, steel yield strength and young modulus 500MPa and 210 GPa respectively. The objective of the research work was to strengthen under-reinforced wall with the help of CFRP band and to find out its significance in the strengthening of the existing structure designed, irrespective of the Eurocode recommendations.

The first wall specimen, S1, was not retrofitted. This specimen was tested as a control specimen and used to observe the RC wall failure mode. The second wall specimen, labelled SR2, was retrofitted by bonding CFRP bands onto each wall face to improve its shear strength and to control cracking within the wall panel. The adopted external CFRP reinforcement pattern was based on the crack pattern observed in the RC wall load test. The composite reinforcement was made from carbon fabric bands that were 50 mm wide and 0.48 mm thick. The Young's modulus is 105 GPa, and the ultimate strength is 1400 MPa. The mesh anchors utilized here has proved its significance in limiting the intermediate crack debonding at L shape joint [6].

The test specimens were subjected to displacement control lateral loading, with the walls acting as cantilevers. Specimens S1 (RC wall) and SR2 (CFRP + RC wall) were subjected to a lateral displacement at the rate of 1 mm/min with a constant axial compression load equivalent to 110 kN applied at the head beam (axial load ratio of 0.14).

The load displacement curves of specimens S1 and SR2 are shown in Fig. 1. Specimen S1 exhibits plastic deformation as it exceeds the 3 mm limit, whereas specimen SR2 exhibits nonlinear elastic behavior until failure. At the plastic yield point of specimen S1, the loads sustained by it and SR2 were 150 kN and 174 kN, respectively. The increase in strength of 24 kN is due to the CFRP external reinforcement.

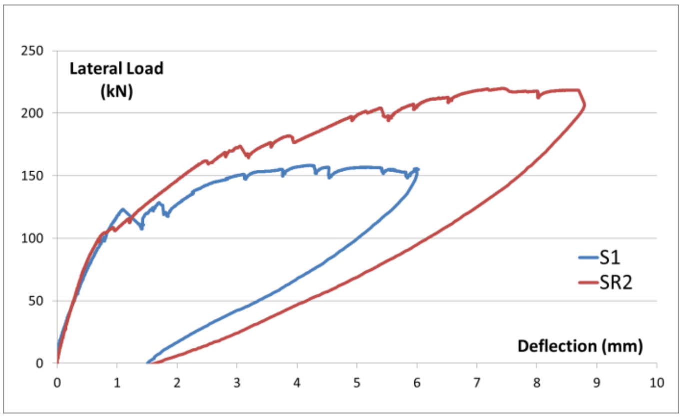

Figure 1. Load displacement curve of specimens S1 & SR2.

3. Modeling

The RC wall was simulated as a two-dimensional strut-and-tie model, as shown in Fig. 2. The dotted lines represent struts composed of concrete only, the solid vertical lines represent ties consisting of steel rebar and the horizontal solid line at the top represents a rigid bar. The most crucial step in strut-and-tie modeling is deciding the load path and the arrangement of the struts and ties. The strut-and-tie pattern used here is based on the location of the vertical rebar in the test specimen and the static load induced at the top of the wall (load end). To make the model as simple as possible, (a) the concrete cover around the tie was not considered because tie concrete covers do not contribute to the resistance and are provided mainly for tension stiffening, particularly under service loads [7]; (b) the horizontal reinforcement was taken into account by fixing ties at the lower end and its equivalent cross sectional area was included in vertical ties; and (c) the vertical load induced on the test specimen was considered in the model while only defining the dimensions of the concrete strut. In case of RC wall strengthened with external CFRP bonding, the tie bars were replaced after reaching the steel yield limit with a composite bar that had a cross-sectional area equivalent to the CFRP band used.

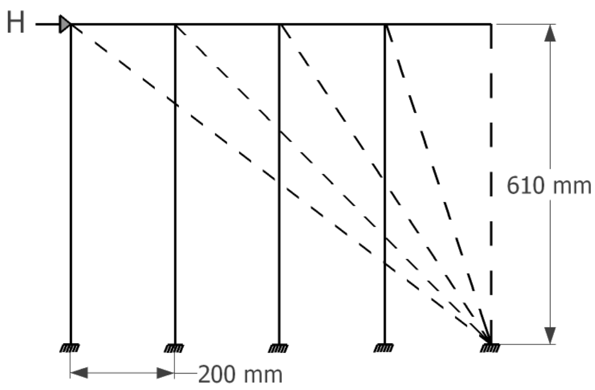

Figure 2. Strut and tie arrangement of the RC short wall model.

The width of the struts was taken to be equal to the width of the wall panel [4,7, 8] and its breadth equivalent to the depth of the compression zone at the base of the wall [8] (Eq. 1).

The struts were shaped as prismatic; therefore, the concrete strength was taken to be $0.85\times f_c^{'}$ [7]. The cross sectional area of the each tie bar was equivalent to four-times the cross sectional area of the steel reinforced bar (∅ 4.5 mm) used with in wall, except at load end tie area was two times ∅ 6 mm reinforced bars. In place of the horizontal tie bars in the model, vertical tie bars were fixed at the bottom and horizontal bar area was added to vertical ties. One single vertical tie was used to depict wall panel reinforcement in the model based on the fact that in case of the short wall subjected to shear failure the orientation of the diagonal cracks are roughly 45° therefore it induce stress in the two orthogonal reinforcement bar simultaneously. In case of CFRP reinforcement, the CFRP tie area was two times the area of the CFRP band used in specimen strengthening. Each CFRP tie bar depicted the two vertical bands bonded on two wall faces. The CFRP bands bonded in transverse direction were meant to keep the vertical bands intact therefore a perfect bond in between CFRP and concrete was assumed in the model. The node width was equal to the width of the wall panel. In the planar dimension of the node at the bottom of the wall's free end, the length and height of the node were equal to as. A CCC-type node was used; therefore, the concrete strength was equivalent to $0.85\times f_c^{'}$ [7].

$$a_s = \left(0.25 + .85\frac{N}{A_w f_c^{'}}\right)\times l_w \tag{1}$$

Here, N = axial force, A_w = net area of the concrete section (wall panel), l_w = length of the wall section in the direction of the shear force and $f_c^{'}$ = compressive strength of the concrete.

Selecting the compression and tension behaviour of materials is an important step in designing because it significantly influences the stress distribution in the section and consequently the values of the internal forces and the specimen global load displacement behaviour. Sargin's law defines the behaviour of concrete under compression. The behaviour of steel is considered to be elastic-plastic and steel hardening is neglected, as illustrated by the presented simplified bilinear curve. The behaviour of the composite plate is considered to be elastic-linear until failure, as shown in Fig. 3.

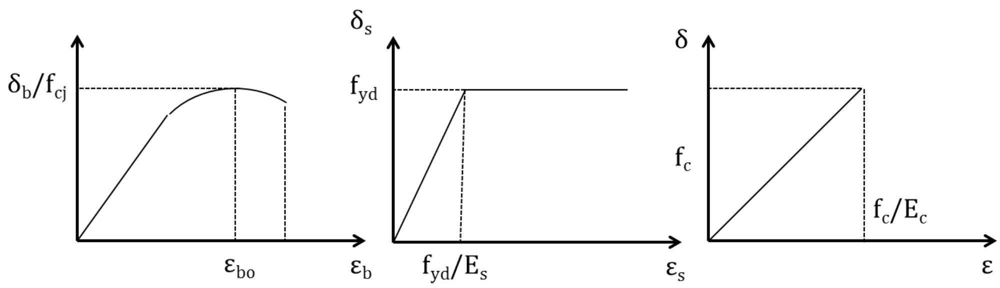

Figure 3. Strut and tie arrangement of the RC short wall model.

Here, ε_{bo}: concrete deformation in compression corresponding to δ_b/f_{cj}; ε_b: concrete ultimate deformation; E_{bj}: concrete initial tangent modulus; E_s: steel rebar modulus of elasticity; f_{yd}: steel rebar design yield strength; E_c: composite modulus of elasticity; and f_c: composite rupture stress.

The adopted analysis pattern is shown in Fig. 4 as a flowchart. First, a lateral load was applied on the truss rigid beam that connected the ties and struts, and then, truss analysis was performed to evaluate the truss elements' axial force (i.e., struts and ties). Based on the considered material properties and the corresponding truss area, strains values in each element were evaluated and were subsequently used to evaluate the resultant deflection at the free end of the truss. In the next step, the stress on the ties was checked. If the evaluated values were less than the steel yield strength limit, the process was repeated by increasing the load, and the Young's modulus of concrete is then modified. If the values were equivalent to the yield strength of the steel, the corresponding tie was replaced with an equivalent constant axial force before the process was repeated. The axial force was calculated based on the yielding force of the tie because the strain hardening of the steel was not considered in the model. If the wall was strengthened with CFRP, the yielded steel ties were replaced with CFRP ties at this stage. The analysis terminated when all the steel tie yields for the RC wall or all the CFRP ties reached their maximum stress levels for the CFRP RC wall. The CFRP maximum stress was taken equivalent to 0.4 % of the CFRP strip ultimate strength.

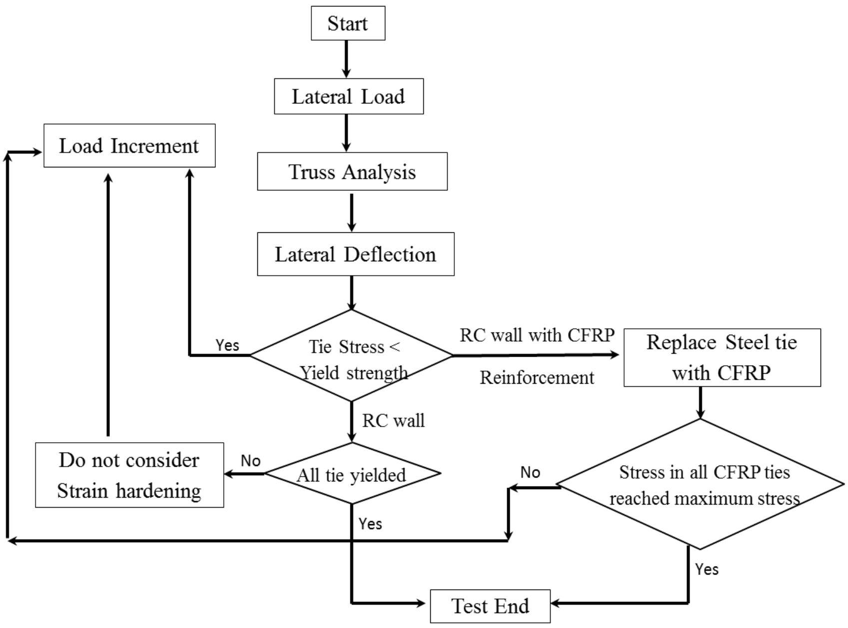

Figure 4. Strut and tie arrangement of the RC short wall model.

4. Results

Figure 5 shows the load displacement curves of specimen S1 (a) based on observed test data and (b) evaluated with the strut-and-tie model. The model accurately represents the load displacement behavior of the tested specimen to a certain extent based on the use of a simplified model. The model well predicted the specimen yielding and its ultimate capacity. However, the model did not sufficiently forecast the initial stiffness and deformability of the specimen. The model behavior was controlled by the behavior of the tie bar because the strut dimensions used were larger, and therefore, the compressive stresses in the strut were considerably less than its strength. The strut-and-tie model predicted that the ultimate capacity for specimen S1 was 148.8 kN, whereas the actual value was 157.8 kN.

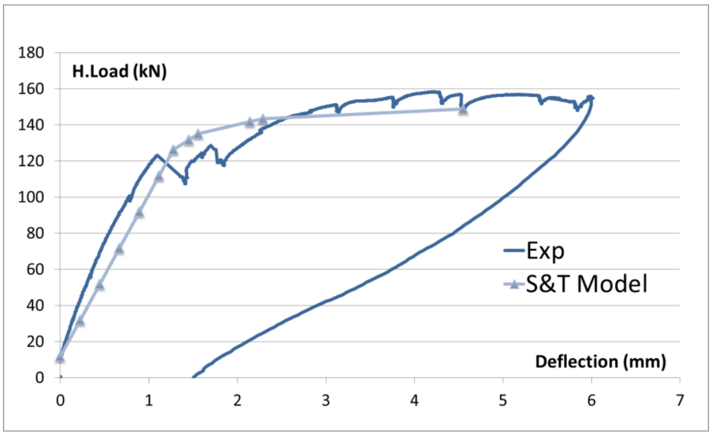

Figure 5. Specimen S1 load displacement curves: experimental and strut-and-tie model.

Figure 6 presents the load displacement curves of specimen SR2 (a) based on measured test data and (b) evaluated with the strut-and-tie model with the difference in the tie bar used. To evaluate the contribution of external CFRP reinforcement, tie bars with different configurations were used in this model. In the first case depicted by a line curve with a circular marker, the tie that represented the steel rebar used in the model of specimen S1 was used, and as the steel yielded, it was replaced with the one that represented the composite material. In this case, the evaluated curve showed coherence, particularly in the interval of 110-160 kN, but then deviated from the experimentally observed curve. In this case, the slipping of CFRP observed during the experiment was negated, and the ties were assumed to use the full strength of the CFRP. In the second case, depicted by a line curve with square markers, the CFRP slip phenomenon was considered by considering the maximum stress observed in the CFRP strip during the experiment as the CFRP ultimate capacity (316 MPa). In this case, the evaluated curve mostly agreed with the experimental curve and well depicted the deformability of the test specimen. However, the ultimate load evaluated was 185 kN, and the experimental value was 218 kN.

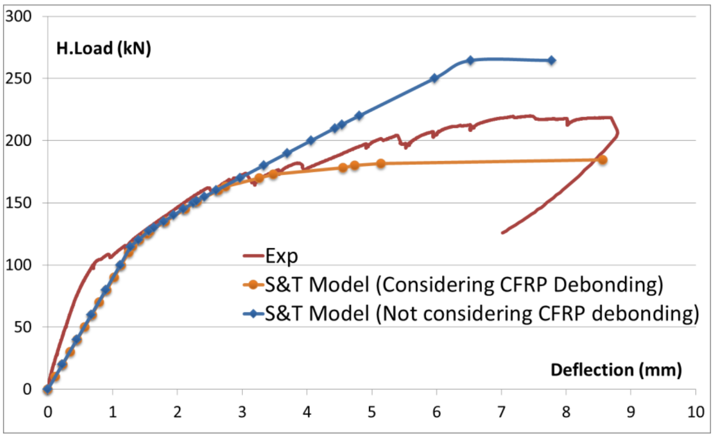

Figure 6. Specimen SR2 load displacement curves: experiment and strut-and-tie model.

5. Conclusion

The proposed simplified strut-and-tie model successfully predicted the load response behavior of short wall specimens with reinforced concrete and reinforced concrete strengthened with carbon fiber-reinforced polymer. The crucial step in this model is defining the load path and assuming a strut-and-tie pattern. The positions of the struts were based on the lateral load applied at the top of the wall, and the induced vertical load was considered in finalizing the dimensions of the strut. The specimens' reinforcement was modeled as vertical ties, and the horizontal reinforced bar effect was considered by fixing the lower end of each tie. In specimens strengthened with CFRP, the tie bars first consisted of a steel rebar similar to the RC wall specimen, and as the stress in the tie bar reached the yield strength value of the rebar, the steel tie was replaced with a CFRP tie. This arrangement was based on the assumption that CFRP reinforcement begins to contribute after the steel rebar yields and the concrete starts cracking. The evaluated load displacement curve was similar to the experimentally measured curve in both cases. However, in the case of the CFRP strengthened specimen, the curve showed coherence with experimentally observed data when the ultimate capacity of the CFRP was reduced to the experimentally observed maximum stress in the CFRP band.

6. References

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