Modeling and Simulation of Reinforced Concrete Shear Wall under Bidirectional Earthquake Action

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Abstract. Reinforced concrete shear walls under bidirectional earthquake action were taken as research object, and discretized into irregular polygons in plane, which were stretched into prisms along thickness direction of the wall. Multiple concrete spring points were distributed at side edges of the prism, deformation of concrete spring points were separately considered in axial directions and tangential directions, and they were mutually coupled in stiffness and strength. Reinforced spring points were arranged at intersection points between reinforcements and polygon prisms, and dowel action of reinforcements were neglected. Through a comparison between numerical simulation results and model test results, it could be seen that numerical simulation could accord with model test very well in aspects of failure mode and load-displacement curve. Axial compression ratio and out-of-plane deformation were taken as parameters to expand test data. Research found that when axial compression was not large, increase of axial compression ratio improved bearing capacity of the specimen. If in-plane force was conducted after a certain displacement out-of-plane was applied, the greater the out-of-plane displacement above the shear wall was, the more the in-plane bearing capacity reduced, and the more in-plane deformability degraded.

1. Introduction
Shear wall is an important load-bearing and lateral load resistance member in modern high-rise buildings, and establishing an accurate and reliable numerical model of shear wall is of great significance to realizing simulation of collapse process of high-rise buildings under earthquake action [1]. Nowadays most researches have established static elastic-plastic analysis macro-model of reinforced concrete shear wall based on finite elements [2] [3] [4] [5]. Some methods like SDOF (single degree of freedom) compensation method and minus-load method have been used to do descending-branch calculation [2], or existing mass point-truss model and multi-montant model have been improved [3] [4], or one new element form [5] has been used to finally realize whole-process simulation of in-plane failure of shear wall. Thus it can be seen that some research results have been achieved on numerical methods about failure process of reinforced concrete shear wall, but few researches have considered out-of-plane influences so that theoretical model can’t totally accord with actual stress state of shear wall. In order to improve shock resistance of short pier shear wall under stress in non-engineering axial direction, concealed bracings are added and anti-seismic static test of short pier shear walls was implemented under stress in non-engineering axial direction (directions forming 45° and 135° angles with engineering axis) in the wall, but it’s only limited to engineering test [6]. In literature [7], the shear wall tests loaded
along 25 and 45 degrees are designed respectively. It is found that with the increase of loading angle, the failure form of shear walls transit from bending shear failure to bending failure, and the flexural crack increases gradually and the plastic hinge is more obvious. In this paper, based on discrete element method, a refined modeling method was proposed, and comparison between numerical simulation result and test result indicated that failure mode and load-displacement curve accorded with each other very well.

2. General method

2.1. Element discretization method of shear wall
When the wall was discretized, a set of discrete growth points was generated in the wall plane (Figure 1a), and Delaunay triangulation network (Figure 1b) was obtained, and a perpendicular bisector of each triangular edge was drawn. Intersection points of all perpendicular bisectors constituted Voronoi graph of discrete point set [7] (Figure 1c and Figure 1d), and this Voronoi graph was taken as bottom edge of the prism, and it was stretched in vertical direction to obtain a prism.

Each side edge of the prism was equally divided into several small area elements, of which the spring point was generated at “centroid” point. And 4 springs were generated in thickness direction in this essay. Figure 2 shows schematic diagram of generation process of prism and spring points. The prism rotated around its own axis under in-plane shear effect, and it would rotate in the direction vertical to the axis under out-of-plane flexural effect.

![Figure 1. Generation process of voronoi graph](image1)

![Figure 2. Polygonal stretching and generation of concrete spring points](image2)

2.2. Reinforcement steel fabric and reinforced spring points
For reinforcement steel fabric actually existing in shear wall (as shown in Figure 3), vectors were formed from starting point to terminal point according to geometric position of each reinforcement during modeling process so as to generate spatial vector reinforcement fabric. When one reinforcement had
intersection point with concrete element, reinforced spring points were generated at this intersection point as shown in Figure 4. In this way, with difference of block size during element division process, one wall block might not have any reinforced spring point or might have multiple reinforced spring points.

2.3. Deformation calculation of concrete spring

The middle plane of two adjacent elemental areas was taken as reference plane for spring deformation calculating. Projection of spring deformation vector in normal direction on the reference plane was taken as axial deformation vector and projection on reference plane was taken as shear deformation vector as shown in Figure 5.

Its set that normal vectors of two planes connected by springs were respectively $n_1$ and $n_2$, then normal vector of reference plane $n$ was:

$$n = n_1 + n_2$$

(1)

Spring deformation vector could be written as $S_{AB}$, and projection vector of which on reference plane was $s$, with normal vector $a$. Figure 6 reflects calculation method of axial and tangential components under different deformation modes of concrete springs. As shown in Figure 6c, under simultaneous axial and tangential deformation, axial deformation $|S_{AB}|\cos \theta$ and shear deformation $|S_{AB}|\sin \theta$ could be obtained through spring deformation vector $S_{AB}$. 

Figure 3. Reinforcement fabric

Figure 4. Locations of reinforced spring points

Figure 5. Deformation calculation of concrete spring
2.4. Representative length of concrete spring

The relationship between spring internal force and its deformation could be obtained through conversion of uniaxial constitutive relationship of concrete. It’s assumed that centroids of two adjacent elements were respectively \( O_A \) and \( O_B \), and the simplest way of taking value of representative length \( L_0 \) was as shown in Figure 7a. Representative length \( L_0 \) was taken as distance between centroids of two elements when spring at interface of two elements represented spring area with thickness being \( t \) and width being \( B \). However, this method would result in material reutilization and distortion. Hence, centroids were connected to angular points to form virtual triangles as shown in Figure 7b, and area of each material represented by spring was sum of areas of two triangles. In this way, no material would be reutilized. Then area of the two triangles was equivalent to a rectangle with width \( B \) and length \( L_0/2 \), namely representative length \( L \) of spring.

3. Material constitutive

3.1. Geometrical physical properties of concrete spring

As shown in Figure 8, concrete spring was decomposed into two parts. One part represented axial performance and the other part represented tangential performance, and the two were mutually coupled. Shear would reduce compressive strength and compression within a certain range would improve shear strength.
3.2. Axial constitutive relationship of concrete spring

Axial Force and deformation of concrete spring could be expressed as equation (2).

\[
P_{ca} = A_c \sigma_c \\
d_{ca} = L \varepsilon_c
\]  

(2)

In the equation, \( P_{ca} \) is axial force of concrete spring, \( A_c \) is representative area, \( \sigma_c \) is axial stress, \( d_{ca} \) is axial deformation and \( \varepsilon_c \) is normal strain.

Axial stress-strain skeleton curve of concrete spring referred to model [8] suggested by Li as shown in Figure 9. In the figure, \( \varepsilon_{c0} \) and \( \varepsilon_{cu} \) are respectively strain and ultimate compressive strain corresponding to ultimate compressive strength of concrete, \( \varepsilon_t \) is tensile stress corresponding to ultimate compressive strength, \( f_t \) and \( f_c \) are respectively tensile strength and compressive strength of concrete, and \( E_c, E_{c1} \) and \( E_{dec} \) are respectively tangent moduluses in elastic branch, strengthened branch and descending branch. When values are given to concrete spring points, \( f_t \) is taken as \( f_t = 0.1 f_c \).

3.3. Tangential constitutive relationship of concrete spring

It’s assumed that before failure of concrete spring, shear behaviors in any direction on the shear surface were identical and unrelated to historical shear stresses in other directions, namely axial loading path didn’t have any influence on shear resistance performance before material failure. So, when tangential deformation was generated in concrete spring, tangential resilience and tangential deformation of concrete spring can be expressed as equation (3):

\[
P_{ct} = A_c \tau_c \\
d_{ct} = L \gamma_c
\]  

(3)

In the equation, \( P_{ct} \) is tangential force of concrete spring, \( \tau_c \) is shear stress, \( d_{ct} \) is tangential deformation and \( \gamma_c \) is shear strain.

Tangential skeleton curve of concrete spring was taken as bilinear model as shown in Figure 10. The curve was linear before maximum shear stress and shear modulus was taken as 0.4 times of initial elasticity modulus. After maximum shear stress, shear stress would be taken as frictional force, which changed with axial compressive force, and friction coefficient was taken as 0.55 [9].
3.4. Axial and tangential coupling of concrete spring

In order to discuss about mutual relationship of concrete between axial stiffness and tangential stiffness under different stress states, concrete material block as shown in Figure 11a was established in finite element software. Block dimension was 100mm×200mm×300mm, uniaxial compressive strength of the material was 30MPa and compound constitutive relationship of concrete in axial direction and shear direction was studied. Numerical test was compressive-shear equivalent displacement load test, namely axial (Y direction in Figure 11a) displacement and tangential (XZ in-plane direction in Figure 11a) displacement on block top maintained identical at every moment until specimen failure.
It could be seen from Figure 11b and Figure 11c that under axial and tangential equal-displacement loading process, compressive peak point and shear peak point would be lowered because of existence of the other party, while slope of curve before shear peak point basically remained unchanged, and slope of curve before compressive peak point also basically didn’t change. Therefore, it could be assumed that initial compressive elasticity modulus under compressive-shear stress state was still $E_c$ and initial shear elasticity modulus under compressive-shear stress state was still $0.4E_c$.

Axial and tangential strength coupling relationship of concrete could be as shown in Figure 12.

3.5. Deformation calculation of reinforcement spring

Shear resistance and dowel action of reinforcement spring were neglected but only compressive and tensile properties along direction of the reinforcement were taken into consideration. So projection of spring deformation vector on reference plan was not taken into consideration in deformation calculation, but instead, deformation $|S_{AB}|$ of spring deformation vector was directly taken as deformation of reinforcement spring, and it was positive under tensile stress and negative under compressive stress.

3.6. Properties of reinforcement spring

Reinforcement spring represented reinforcement mechanical properties within each half length in elements at two spring ends, and reinforcement lengths in elements at each end were as shown in Figure 4. Resilience and deformation of reinforcement spring could be expressed as equation (4):

$$
p_s = A_s \sigma_s, \quad d_s = L \varepsilon_s
$$

(4)

In the equation, $p_s$ is force of reinforcement spring, $A_s$ is representative area, $\sigma_s$ is stress, $d_s$ is deformation and $\varepsilon_s$ is normal strain. Stress-strain relationship of reinforcement spring was trilinear model as shown in Figure 13.

3.7. Strength value of concrete spring

According to standard dimension of prism specimen block in concrete material characteristic test, a compressive-resistance numerical model with width and height respectively being 150mm and 300mm was established. Maximum spacing of generatrixes during random division was taken as 25mm, and other parameters were: elasticity modulus 21,000MPa, Poisson’s ratio 0.21 and time step $10^{-5}$s. Fractal geometrical theory [10] was used to do a trial calculation of rules between element strength value and average strength value of standard specimen, and it’s obtained that: when distance between generatrixes was 25mm, to obtain 26.48MPa compressive strength of macro specimen, the element strength should be taken as $f_{ce,25}=59.2$MPa.

![Figure 12. Failure criterion of concrete spring [11]](image)

![Figure 13. Stress-strain relationship of reinforcement spring](image)
4. Loading and output
During reciprocal loading process of shear wall, failure conditions in two directions were quite approximate. Therefore, when discrete element model of shear wall was established, only monotonic loading situation was taken into consideration, and contact and friction between broken faces after element failure were neglected. Bottom foundation beam was regarded as an integral element which maintained static and top loading beam was also regarded as an integral element, which bore vertical load and horizontal load and occured rigid motion. During loading process, firstly a certain vertical displacement was applied on loading beam, and elements started to move and rotate until whole system reached iterative balance. Vertical displacement was accumulatively added when spring resultant force of the loading beam reached anticipation in vertical component, and horizontal displacement (or force) was applied to the loading beam, this horizontal displacement (or force) could be in-plane or out-of-plane or inclined. Through trial calculation, 0.01mm was taken as displacement value of loading of each step when unit systems were selected as mm, kg and s.

When a loading was completed and accelerated speed and speed of each element in the system were extremely low, it’s believed that the system reached a balanced state. Horizontal displacement value of the loading beam and component of spring resultant force in horizontal direction were recorded, namely a numerical point on member load-displacement curve.

5. Model test

5.1. Specimen design
There were 3 groups of specimens designed. Each group contained 3 specimens and total specimen list was as shown in Table 1. Length of embedded column section along the direction of wall was 180mm, 4ø8 longitudinal bar was used in embedded column, longitudinal bar reinforcement ratio was 1.478% and ø6.5@100 as reinforcement stirrup was used in embedded column. Concrete compressive strength $f_c$ of C30 was 26.48MPa, peak strain $\varepsilon_{c0}$ 0.0016, ultimate strain $\varepsilon_{cu}$ 0.0076 and elasticity modulus $E_c$ 2.76×10^4Mpa. Measured yield strength of reinforcement $f_y$ was 292.6MPa, ultimate strength $f_u$ was 374.6MPa, maximum ductility ratio $A_{gt}$ was 9% and elasticity modulus $E_s$ was 2.2×10^5MPa.

| Specimen No. | Loading angle | Sectional size mm² | Shear span ratio | Concrete Strength | Vertical reinforcement | Horizontal reinforcement | Axial compression ratio |
|--------------|---------------|---------------------|------------------|-------------------|-----------------------|-------------------------|------------------------|
| W1-1         | 0°            | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.10                   |
| W1-2         | 0°            | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.21                   |
| W1-3         | 0°            | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.31                   |
| W2-1         | 45°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.10                   |
| W2-2         | 45°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.21                   |
| W2-3         | 45°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.31                   |
| W3-1         | 90°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.10                   |
| W3-2         | 90°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.21                   |
| W3-3         | 90°           | 900×80              | 0.852            | C30               | ø6.5@180              | ø6.5@250                | 0.31                   |

Calculation basis of specimen cracking load $V_{cr}$ was obtained according to Literature [16] and [17]. Loading was conducted according to China’s industrial standard Regulations JGJ101-15 [18]. Load increment, displacement increment and cycle index during specimen loading process were as shown in Table 2.

5.2. Loading scheme
Loading devices of specimens under different loading angles were as shown in Figure 14.
For specimens under in-plane loading in W1 group, in-plane shear resistances of specimens were calculated respectively according to methods given by ACI318 [12], ASCE-ACI 426 committee [13], Liu Hang [14], Alfredo [15] and China's Nation Standard GB50010 [16], and the maximum value was taken as corresponding horizontal counter force of the tester under specimen failure. For specimens under out-of-plane loading in W3 group, out-of-plane flexural bearing capacities of specimens were calculated through GB50010, and corresponding horizontal force was taken as estimated maximum horizontal counter force of the tester. For specimens under loading along 45° in W2 group, it’s assumed that wall failure under loading process was controlled by out-of-plane flexural bearing capacity, and then horizontal counter force needed by corresponding specimens in W2 group could be obtained through estimated horizontal counter force of specimens in W3 group (90°).

![Test loading devices](image)

**Figure 14.** Test loading devices

**Table 2. Load increment and cycle index of specimens**

| Specimen No. | Loading angle | Axial compression ratio | $V_{cr-f}$ /kN | Estimated $V_{oe}$ /kN | Preloading /kN (Cycle index) | Load increment before cracking /kN (Cycle index) | Load increment before yield /kN (Cycle index) | Displacement increment /mm (Cycle index) |
|--------------|---------------|------------------------|----------------|------------------------|-------------------------------|-----------------------------------------------|--------------------------------------------|---------------------------------|
| W1-1         | 0°            | 0.10                   | 56.5           | 279.9                  | 17(2)                         | 22(3)                          | 25(1)                                      | 1(3)                             |
| W1-2         | 0°            | 0.21                   | 56.5           | 331.4                  | 17(2)                         | 22(3)                          | 28(1)                                      | 1(3)                             |
| W1-3         | 0°            | 0.31                   | 56.5           | 366.2                  | 17(2)                         | 22(3)                          | 32(1)                                      | 1(3)                             |
| W1-4         | 0°            | 0.06                   | 73.9           | 386.8                  | 22(2)                         | 30(3)                          | 36(1)                                      | 1(3)                             |
| W2-1         | 45°           | 0.10                   | 7.4            | 34.8                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W2-2         | 45°           | 0.21                   | 7.4            | 58.2                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W2-3         | 45°           | 0.31                   | 7.4            | 73.0                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W2-4         | 45°           | 0.06                   | 9.7            | 55.2                   | 3(2)                          | _                             | _                                          | 1(3)                             |
| W3-1         | 90°           | 0.10                   | 5.3            | 24.6                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W3-2         | 90°           | 0.21                   | 5.3            | 41.2                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W3-3         | 90°           | 0.31                   | 5.3            | 51.6                   | 2(2)                          | _                             | _                                          | 1(3)                             |
| W3-4         | 90°           | 0.06                   | 6.8            | 39.1                   | 2(2)                          | _                             | _                                          | 1(3)                             |

Notes: 1. Under W1 in-plane loading process, $V_{cr-f}$ expresses corresponding horizontal load value when bending shear cracks appear; during W2 inclined loading and W3 out-of-plane loading process, $V_{cr-f}$ expresses corresponding horizontal load value when bending cracks appear; 2. Displacement loading is adopted for W2 inclined loading and W3 out-of-plane loading.
6. Bidirectional stress simulation test of shear walls

6.1. Comparison of discrete element model and test
C++ language was used to compile irregular division mode to simulate shear walls mentioned above. Specimen size and reinforcement information were completely consistent with test, and displacement loading was adopted.

6.2. Comparison between calculation result and test result of shear walls under in-plane loading
Comparison of failure modes of specimens under in-plane loading is as shown in Figure 15. It could be seen from the figure that failure modes under different axial compression ratios accorded with test results very well. As axial compression ratio increased from 0.10 to 0.30, inclined angle between inclined crack and horizontal plane gradually enlarged, and width of stripe under inclined concrete compression was firstly widened and then narrowed.

Comparison of horizontal load-displacement curves of shear wall is as shown in Figure 16. It could be seen that for shear walls under different axial compression ratios, discrete element model could be used to obtain initial stiffness and peak shear force reasonably, but displacement value corresponding to peak point needed improvement. The reason was that shear failure process of shear wall was an extremely complicated strong-nonlinearity process involving cracking, closing, friction, contact and slippage, but the process was much simplified during numerical simulation process. For example, contact was not taken into consideration and reinforcement slippage was neglected, which would influence overall performance of shear wall after cracking. So, further perfection and improvement were still needed.

6.3. Comparison of calculation result and test result of shear walls under out-of-plane loading
Direction of displacement load on the above loading beam was changed into out-of-plane direction to simulate shear walls under out-of-plane stress. Comparison of failure modes was as shown in Figure 17. Thus it could be seen that failure modes of shear walls under out-of-plane stress obtained through numerical test accorded with failure modes obtained through model test.

![Figure 15. Comparison of failure modes of shear walls under in-plane loading](image)
a) Axial compression ratio 0.10                       b) axial compression ratio 0.21

c) Axial compression ratio 0.31

**Figure 16.** Comparison of horizontal load-displacement curves of shear wall under in-plane loading

In addition, through a comparison of load-displacement curves under out-of-plane stress in Figure 18, it could be seen that discrete element simulation results could reflect performances of shear walls under out-of-plane flexural stress very well and discrete element could accurately simulate linear branch, strengthened branch and descending branch of load-displacement curves. As axial force during actual loading process was vertical applied to wall top through a jack, the greater vertical force was, the greater frictional force between roller on jack top and loading beam was. Thus, load-displacement curve
obtained through test was higher than that obtained through numerical simulation, and the higher the axial compression ratio, the most obvious the phenomenon.

Figure 18. Comparison of load-displacement curves of shear walls under out-of-plane loading

6.4. Comparison between calculation result and test result of shear wall under inclined loading

It could be seen from the comparison of failure modes of shear wall under 45° loading in Figure 19 that failure mode obtained through numerical test coincided with failure model obtained through model test, and both failures happened in out-of-plane direction.

In the model test, as in-plane displacement was extremely small, only out-of-plane load-displacement curves of the wall were compared. Through a comparison of out-of-plane load-displacement curves of shear wall under 45° loading as shown in Figure 20, it could be seen that form of load-displacement curves obtained through model test approached model test result, and the reason why curves could not totally accorded with each other lied in frictional resistance of the loading device. Due to existence of frictional force from the device, when axial compression ratio increased from 0.1 to 0.3, reaction force measured in the test became greater and greater when compared with numerical simulation result.
6.5. Influence of Out-of-plane Displacement of Shear Walls on In-plane Shear Behaviors

For more comprehensive understanding of performance of shear wall under actual operating state, another loading condition was designed, namely a certain out-of-plane displacement was applied above the wall, then the out-of-plane displacement was fixed and horizontal displacement was applied in plane so as to understand in-plane shear behaviors under different out-of-plane stress states. Here numerical model remained unchanged, only loading condition changed. Results were as shown in Figure 21.
It could be seen clearly that, for a certain displacement applied out of the plane, when axial compression was not too large, increase of axial compression ratio improved bearing capacity of the specimen. In-plane mechanical properties would change when a certain displacement was applied out of the plane. The greater the out-of-plane displacement above the shear wall, the more the in-plane bearing capacity reduced, and the more in-plane deformability degraded.

7. Conclusion
It’s found through simulation analysis, on the precondition of element simplification into rigid body, computer model using irregular division method could simulate inclined cracks similar to those in the model test. Multiple spring points were generated on side face of the prism to simulate stress state of the
wall under out-of-plane flexural loading. Reinforcing in the wall was represented by a group of vectors according to actual location of reinforcement, from which intersection points of steel vectors and element side faces were found out and these intersection points were namely locations of reinforcement spring points on each element. Through a comparison between numerical simulation results obtained through this refined modeling method and model test results, it could be seen that numerical simulation could accord with model test very well in aspects of failure mode and load-displacement curve. Therefore, the modeling method in this paper could be an effective supplemental means of experimental research on shear wall.

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