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Seismic behaviour of lightly reinforced concrete structural walls with openings

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An experimental investigation was carried out to examine the seismic behaviour of three lightly reinforced concrete walls with openings subjected to reversed cyclic loading. The three specimens consisted of one solid wall as the control specimen while the other two walls were detailed with regular or irregular openings. The test results indicate that all three specimens eventually failed when the outermost reinforcing bars fractured while the concrete in the compression zone crushed and spalled severely near the base. The specimen with five openings had ultimate strength and stiffness degradation similar to the control specimen. The specimen with nine openings had a lower ultimate strength but exhibited higher ductility, slower stiffness degradation and a more significant shear contribution than the control specimen. Furthermore, strut-and-tie models were developed to predict the ultimate strength of walls with openings. The results obtained from the strut-and-tie models were found to be consistent with the experimentally observed results.

Notation

f'_c	compressive strength of concrete
f_u	ultimate strength of reinforcing bars
f_y	yield strength of reinforcing bars
h_w	vertical distance from lateral loading point to wall base
l_w	width of wall
M_i	nominal flexural strength
V_i	horizontal load at the middle of top beam associated with nominal flexural strength
$\delta_{0.75}$	displacement responding to $\pm 0.75V_i$ cycles
δ_y	yield displacement of the wall

Introduction

Reinforced concrete (RC) structural walls are usually used in buildings to resist lateral loads caused by wind and earthquakes. However, they commonly have some openings for functional or aesthetic purposes. In cases where openings are relatively large or located at critical regions, they may have a significant influence on the strength, stiffness and ductility of the RC wall. The Architectural Institute of Japan (AIJ, 2004) limits the minimum strength reduction factor of a structural wall due to the openings to 0.6 by restricting the maximum ratio of opening dimensions to the corresponding wall length and height. Most previous works (e.g. Aktan and Bertero, 1984; Choi *et al.*, 2012; Mo and Shiau, 1993) carried on structural walls have focused on the behaviour of walls with or without symmetric openings, but a few studies

on walls with irregular openings have been reported (Li and Chen, 2010; Wang *et al.*, 2012). Wood (1991) concluded that walls with irregular openings performed satisfactorily in severe seismic regions and Ali and Wight (1990) and Yanez *et al.* (1991) indicated that such walls would behave in a manner similar to a solid wall provided they were well detailed. Experimental studies conducted by Yanez *et al.* (1991) on the seismic performance of six walls with irregularly distributed openings and reinforced with small or moderate amounts of steel that were designed using strut-and-tie models and the capacity design criteria indicated that pierced walls could exhibit significant ductility and also suggested that the strut-and-tie approach could be a robust tool for the evaluation of slender walls with openings. Strut-and-tie models (Maxwell and Breen, 2000; Sagaseta and Vollum, 2010; Schlaich *et al.*, 1987) have been used intuitively for many years in the design of concrete structures, whereby complex stress fields inside a structural member arising from applied loads are simplified into discrete compressive and tensile force paths. With the aid of strut-and-tie models, better visualisation and understanding of the distribution of internal forces and the mechanism of force transfer can be achieved. Taylor *et al.* (1998) carried out experimental and analytical studies on slender RC structural walls with an opening at the base of each wall near the boundary; experimental results showed that the walls displayed stable hysteretic behaviour and significant ductility. The strut-and-tie model was also found to be an effective tool for the design of the discontinuous region.

The main objectives of this paper are twofold. First, an experiment for investigating the seismic behaviour of lightly reinforced concrete structural walls with openings that are regularly and irregularly distributed will be presented. Second, comprehensive strut-and-tie models will be developed to illustrate the main load paths in stress flow patterns and used to formulate a rational approach to predict the theoretical strength of wall specimens with openings.

Experimental programme

Wall details

The geometry and reinforcement details of three specimens are illustrated in Figure 1. Each specimen consisted of three sub-assemblies – the top beam, the web and the foundation beam. Specimen W1 (solid wall) was designed as the control specimen; it was 2600 mm wide, 2300 mm high and 120 mm thick, with an aspect ratio of approximately $h_w/l_w = 0.98$. Specimens W2 and W3 had the same profile dimensions as W1, but W2 had five irregularly arranged openings and W3 had nine regularly arranged openings. The vertical and horizontal reinforcement ratios in the web were 0.60% and 0.51% respectively. In addition, mild steel bars were added in other regions (e.g. the panel zones) to limit cracking of the concrete.

Material properties

Ready-mix concrete with $f'_c = 30$ MPa, 13 mm maximum aggregate size and a slump of 100 mm was used to cast the specimens. The measured compressive strengths of specimens W1, W2 and W3 were 38.1 MPa, 38.3 MPa and 39.1 MPa respectively. The two types of steel reinforcing bars used for all specimens were high-yield steel bars (hot-rolled) with a nominal yield strength of 460 MPa and hot-rolled mild steel bars with a nominal yield strength of 250 MPa. The tested strengths of the steel bars (T10, R10 and R6) are shown in Table 1.

Testing procedure and instrumentation

The test rig used is shown in Figure 2. The foundation beam of the specimen was fully fixed whereas the top of the specimen was free to move. Load to the wall specimens was applied through a hydraulic jack with a capacity of 2000 kN in compression and 1200 kN in tension. Reversed cyclic lateral loading was applied to the top beam of each wall in accordance with the loading sequence recommended by Park and Paulay (1975).

The loading sequence began with three force-controlled cycles, which assumed that the structure was still within the elastic range, followed by displacement-controlled cycles in which the displacement was increased according to the displacement ductility factor (DF). In the force-controlled cycles, the horizontal loadings increased from $\pm 0.25V_i$ to $\pm 0.75V_i$ in steps of $\pm 0.25V_i$, V_i being the horizontal load at the middle of the top beam associated with the nominal flexural strength (M_i) being reached at the critical sections of the walls. According to the top drift $\delta_{0.75}$ obtained from $\pm 0.75V_i$ cycles, δ_y can then be calculated

from $\delta_y = \delta_{0.75}/0.75$. The displacement-controlled loading process began with one cycle assuming $DF = \pm 1$, followed by the two cycles in each successive DF, which was $DF = \pm 2, \pm 3, \pm 4, \dots$, where DF is calculated by $DF = \delta/\delta_y$. It is worth noting that failure is typically assumed to occur when the lateral load drops below 80% of the maximum value attained in previous cycles. However, in this study, the wall was not loaded until the concrete near the base crushed or spalled severely.

A load cell of capacity 2000 kN was connected to the hydraulic jack to measure the applied shear force. Three types of linear variable differential transducers (LVDTs), with 300 mm travel, 100 mm travel and 50 mm travel, were applied to measure top drift, flexural deformations and shear deformations (Wu, 2002). LVDTs with 300 mm travel were attached to the middle of the end of the top beam to measure top drift. Figure 3 shows the distribution of transducers in each specimen. In addition, FLA-type 5 mm gauge length strain gauges were used to measure local strains in the steel reinforcing bars.

Test results

Crack patterns and failure modes

Crack propagation patterns for specimens W1, W2 and W3 are shown in Figures 4 to 6 respectively. All walls eventually failed when the outermost reinforcing bars fractured while the concrete in the compression zone crushed and spalled severely near the base.

At $0.50V_i$, specimen W1 was in the elastic range and no cracks were observed. At $0.75V_i$, flexural cracks appeared on the bottom half web. At $DF = \pm 1$ (Figure 4(a)), no new major cracks formed but the existing cracks extended into long diagonal cracks. At $DF = \pm 2$, new major cracks formed at the upper half of the web; these cracks had a shorter horizontal length and developed into diagonal cracks quickly. Existing cracks propagated where the crack width measured up to 2.5 mm. Many minor cracks formed at the bottom of the web near the joint with the foundation beam. At $DF = \pm 3$ (Figure 4(b)), the measured crack width was up to 6.0 mm. Crushing and spalling of the concrete cover was observed. As a result, one of the vertical reinforcing bars at the right-hand bottom edge of the web buckled at the second cycle. A slipping face, approximately 100 mm above the joint to the foundation beam, clearly formed. At $DF = \pm 4$, all major cracks formed and deformations were concentrated along the sliding face. A vertical reinforcing bar at the left-hand bottom edge of the web buckled and more bars buckled on the right. A large piece of concrete started to spall. At $DF = \pm 5$ (Figure 4(c)), a large piece of concrete separated at the right-hand bottom edge of the web, leaving the reinforcing bars located here exposed. Three pairs of reinforcing bars at this location buckled. At $DF = \pm 6$ (Figure 4(d)), the outermost pair of reinforcing bars fractured at the right-hand bottom edge of the web on the second cycle.

Specimen W2 behaved similarly to W1; it was in the elastic range and no cracks were observed at $0.50V_i$. At $0.75V_i$, only a

Figure 1. Geometry and reinforcement details of specimens W1, W2 and W3 (dimensions in mm)

	f_y : MPa	f_u : MPa
R6	308.4	428.3
R10	385.1	502.0
T10	480.0	545.1
T13	492.5	580.9
T20	512.0	606.6

Table 1. Measured steel bar properties

Figure 2. Test-rig assembly

few horizontal cracks were noted in the two short columns at the bottom edges of the web. At $DF = \pm 1$, shown in Figure 5(a), flexural cracks appeared at a closer spacing with a shorter developed length compared with specimen W1, and then they extended to form inclined diagonal cracks that passed through the openings without changing their angle of inclination. At $DF = \pm 2$, the major cracks were fully developed and symmetrical cracking patterns were achieved under both negative and positive loading cycles. The measured crack width was up to 3.0 mm. At $DF = \pm 3$ (Figure 5(b)), the measured crack width was up to 8.0 mm. Cracking in the bottom panel zone between three openings was extensive with many minor cracks. Crushing and spalling of the concrete cover were observed. As a result, a vertical reinforcing bar in the column at the right-hand bottom edge showed signs of buckling during the second cycle. At $DF = \pm 4$ (Figure 5(c)), deformation was concentrated on the two short columns at the bottom edges of the web where crushing and spalling of the concrete were very severe. For the right-hand column, three pairs of vertical reinforcing bars were exposed with obvious buckling. For the left-hand column, the outermost pair of vertical reinforcing bars and one stirrup were exposed. At $DF = \pm 5$ (Figure 5(d)), the two short columns at the bottom edges of the web were severely damaged and their reinforcing bars eventually fractured. The buckling of the reinforcing bars extended to the middle panel zone at the bottom. Two pairs of bars were buckled at the edges of the panel.

Figure 3. LVDT positions of specimens W1, W2 and W3 (dimensions in mm)

Figure 4. Crack propagation for specimen W1

Figure 5. Crack propagation for specimen W2

Figure 6. Crack propagation for specimen W3

Specimen W3 was in the elastic range and no cracks were observed at $0.25V_i$. However, at $0.50V_i$, mainly flexural cracks and a few diagonal cracks started to develop at the beam and column zones. At $DF = \pm 1$, shown in Figure 6(a), more diagonal cracks propagated. Three major parallel diagonal cracks formed on the positive loading cycle. At $DF = \pm 2$, a similar phenomenon occurred in the negative loading cycle. Also, existing diagonal cracks propagated and crack widths up to 3.0 mm were measured. At $DF = \pm 3$ (Figure 6(b)), these cracks became wider with widths up to 10.0 mm. At $DF = \pm 4$, many minor cracks developed in the beam and column zone and the concrete started to crush. At $DF = \pm 5$ (Figure 6(c)), deformation was concentrated along the three sets of major diagonal cracks for both loading

directions mentioned previously. Crushing and spalling of the concrete was severe. Cracks opened so widely that the web was exposed. At $DF = \pm 6$, crushing of the concrete became very severe for the two columns at the bottom edges. Large pieces of concrete separated from the web. Reinforcing bars were exposed and buckling was also observed. At $DF = \pm 7$ (Figure 6(d)), movement was observed along the diagonal cracks. The four columns at the bottom were severely damaged. A large piece of concrete cover spalled away and bars buckled. One reinforcing bar located at the bottom right-hand column was fractured.

The test results indicated that W3 demonstrated a more ductile behaviour than specimen W1 or W2.

Hysteretic response

The experimental lateral load–displacement hysteretic responses of all specimens are depicted in Figure 7. A definite pinching was observed in all specimens. Lower strength was observed for the second cycle for each DF. Specimen W1 reached its peak strength of 400 kN at $DF = \pm 3$ and started to decrease after that. W2 reached its peak strength of about 450 kN earlier at $DF = \pm 2$ and started to decrease thereafter. W3 reached its peak strength of about 350 kN at $DF = \pm 2$. As the ductility level increased, the hysteresis loops flattened, indicating that the stiffness of the wall degraded with the progression of cycles. Both W1 and W2 were considered to have failed when their maximum strengths, developed at $DF = \pm 6$ and $DF = \pm 5$ respectively, were 40% smaller than the recorded peak strength. W3 was considered to have failed at $DF = \pm 7$ when the maximum strength developed at this stage was 47% smaller than the recorded peak strength.

Initial stiffness and stiffness degradation

In order to provide an approximate qualitative measure of stiffness degradation of the specimens, the secant stiffness in each cycle was calculated by connecting the points corresponding to the maximum positive displacement in one-half of the cycle and the maximum negative displacement in the other half of the cycle. The trend of stiffness degradation was traced for each specimen. Figure 8 shows that the secant stiffness of all the specimens decreased exponentially. The initial stiffness of W1 obtained from the test was 60.6 kN/mm. At failure, its stiffness dropped by 15% of the initial amount. For specimen W2, the initial stiffness obtained from the test was 68.3 kN/mm, which was slightly larger than that of W1. It could be concluded that five openings configured on the web (as in W2) did not have any effect on initial stiffness. The secant stiffness of W2 degraded at a similar rate as W1. At failure, its stiffness dropped to 13.9% of the initial value. The initial stiffness of W3 obtained from the test was 50.8 kN/mm, which was 16% smaller than that of W1. The nine regularly placed openings on the web of the wall significantly reduced the initial stiffness. However, the degradation of the secant stiffness of specimen W3 was slower than that of W1. At failure, the stiffness dropped to 11.3% of the initial amount.

Displacement decomposition

The relationship between experimental displacement decomposition against the ductility factor of each specimen is shown in Figure 9 (where on the x -axis the number represents the displacement ductility factor, A = first cycle and B = second cycle). Specimen W3 produced similar results for both positive and negative loading directions, unlike the results obtained from W1 and W2. The flexural component dominated the total lateral drift of W1 and W2. The flexural component of specimen W1 consisted of a small variation in the positive loading cycles of about 60% of the total lateral drift for all cycles. For W2, the first two cycles had more than 70% of the total lateral drift dominated by flexural components while cycles after that had about 70% dominated by flexural response. After $DF = \pm 1$, the flexural component of W3 was smaller than that of W1 or W2; its flexural

Figure 7. Lateral load–top displacement relationship for all specimens

Figure 8. Secant stiffness of tested walls with respect to ductility

contribution was maintained at 40% of the total lateral drift for each cycle. The failure stage of the negative loading cycle saw the flexural component of W1 decrease from 70% to 40% of the total lateral drift.

In both loading directions, the shear component of W1 was at a constant level of 20% of the total lateral drift. The unaccounted portion for W1 in each loading cycle could be due to the observed sliding shear deformation. This is illustrated at $DF = -4$, where the measured deformation component along the bottom sliding face was 20% of the total imposed lateral drift. This reading related well to the displacement decomposition result, which showed that the unaccounted portion at $DF = -4$ was about 17% of total lateral drift. The shear contribution for W2 was more than that of specimen W1. This could be attributed to the reduced shear stiffness of specimen W2 due to its openings. Its shear component ranged from 25% to 30% for a DF of 2 to 4, after which the shear contribution increased to 40% for the last three cycles. The sum of the flexural and shear components was almost equal to the imposed drift for all the loading cycles in negative loading. This finding related well to the observation that no sliding shear deformation occurred in this loading direction in the experiment. W3, on the other hand, had about 60% shear contribution, larger than that in W1 or W2. Its nine regularly placed openings greatly reduced its shear stiffness. The sum of the flexural and shear components for every cycle was equal to or more than the total imposed drift. This proved that W3 had no significant sliding displacement prior to the onset of strength degradation, indicating no premature failure caused by this mechanism.

Strut-and-tie modelling

Main load paths of walls with openings

The non-linear finite-element software UC-Win/Mesh and UC-Win/WCOMD (Forum8, 2000) was used to analyse the main load paths of the three specimens under horizontal loading. This

program has been proved capable of modelling the path-dependent two-dimensional static non-linear analysis of various RC structures. Figure 10 demonstrates the principal tensile and compressive stress flow of W1, W2 and W3. Figure 10(a) shows the stress flow pattern of W1 near failure. The compressive stress flow gives a distinct strut mechanism that directly transfers the load from the loading point to the support and assumes a fan-shape, spreading from the bottom edge of the web. Figure 10(b) shows the principal tensile and compressive stress flow of W2. The five regularly placed openings in W2 regulate the stress flow in the web such that the stress flow follows a definite pattern. A stream of compressive stress flows through panels 1 and 3 and further extends into the right-hand bottom column zone for the positive loading direction. This path of stress flow should be the main mechanism that transfers lateral load to the foundation beam, and it can be seen that bottle-shaped compressive stress flow patterns were developed in the two panel zones. Another compressive stress flow was observed at panels 2 and 4 in the web. The stress flow at panel 4 is in a fan-shaped pattern and connected to a stream of vertical tensile stress flow in the middle of the web. Figure 10(c) shows the principal tensile and compressive stress flow patterns of W3. The evenly distributed openings in the web further regulate the stress flow of W3 as compared with W2. Specimen W3 did not show sophisticated fan-shaped and bottle-shaped compressive stress flow patterns. Instead, a very simple pattern of stress flow prevails directly along the diagonals of the beam and column zones. These stress flows along the diagonals connect to each other and three very obvious paths of load transfer are shown, thus corresponding well with the experimental crack pattern. This pattern of stress flow indicated a direct strut mechanism along the diagonals of the beam and column zones.

Strut-and-tie model for a wall with openings

The design of RC walls with openings is complicated and early design codes provided little guidance towards a rational approach. However, ACI 318-08 (ACI, 2008) gives some provisions on application of the strut-and-tie modelling of structural members. In strut-and-tie models, which have been used intuitively for many years in the design of concrete structures, complex stress fields inside a structural member due to applied loads are simplified into discrete compressive and tensile force paths. The compressive concrete struts and tensile steel ties are joined at the nodal zones. The following assumptions were made in the development of the strut-and-tie models of all specimens.

- All reinforcements were lumped into one tie; its position was located in the centroidal axis of the reinforcement lumped into it.
- The strut position was determined by keeping its concrete compressive stress lower than the strength limitations suggested by Schlaich *et al.* (1987), which are $0.68f'_c$ for a concrete strut with cracks parallel to it or $0.51f'_c$ for a concrete strut with skew cracks.
- The struts were idealised into a prismatic shape instead of a bottle-shaped or fan-shaped stress field.

Figure 9. Displacement component plotted against drift ratio relationship for all specimens

Figure 10. Stress flow patterns of all specimens

- The node was not a problem for tensile and compressive failure.

Lateral force transfer mechanisms in walls with openings were the focus of this study. Therefore, strut-and-tie models for the walls with openings (specimens W2 and W3) were developed; the strut-and-tie model for the solid wall (W1) can be found elsewhere (Li *et al.*, 2015).

Referring to Figure 10(b), the main stress flow paths in W2 can be simplified as in Figure 11, which clearly illustrates the force transfer mechanism of W2 for positive loading cycles. In the strut-and-tie model shown in Figure 11, the dashed lines are struts and the full lines are ties; the imposed lateral load is solely transferred into panel 1 at the top before going into panel 3 and eventually directed to the foundation beam through the right-hand bottom column. Another path is through the bottom panel 4. This stream of compressive stress flow can be taken to be transferred from panel 2, where a tie is connected horizontally for equilibrium. Tables 2 and 3 respectively list the primary properties of the ties and struts.

The strut-and-tie model of W2 was statically indeterminate for any particular imposed load at various loading stages. The theoretical strength of W2 was predicted based on the criterion that the tie at the left-hand bottom column failed. The calculations are illustrated in Figure 11. The theoretical strength predicted was 354.4 kN, which was 79% of the maximum strength from the average of the positive and negative loading cycles in the experiment. The underestimated theoretical strength could be due to the selection of the two most representative stress paths, omitting other possibilities. Consequently, the imposed lateral load shared by each path increased. Also, the nature of the model resulted in two ties being heavily dependent on the angle of inclination of the path that transferred the imposed lateral load to panel 1. However, it was observed that this angle was almost constant in the finite-element stress flow pattern, as shown in Figure 10(b).

The stress flow of W3 is more regular than that of W2 due to the nine evenly distributed openings in the web, as shown in Figure 10(c). In this specimen, sophisticated fan-shaped and bottle-shaped compressive stress flows were not found. Instead, a very simple pattern of stress flow prevails, directly along the diagonals of the beam and column zones. Three very obvious paths of principal compressive stress flow are shown. The crack pattern developed during the positive loading cycles in Figure 10(c) proved this finding from the finite-element analysis. Three sets of long diagonal cracks appeared at the same locations as the paths of compressive stress flow. The pattern of stress flow for specimen W3 indicates a direct strut mechanism along the diagonals of the beam and column zones. Again, with dashed lines used to mark out the direction of these compressive stress flows and full lines for the direction of tensile stress flows, the force transfer mechanism and main load paths in this wall specimen are shown in Figure 12. The strut-and-tie model of specimen W3 is more complicated than that of W2 due to the fact that more paths are available to transfer the imposed lateral load. A comprehensive description of the strut-and-tie model for W3 is given by Zhao (2005). The predicted strength of W3 is 305.0 kN, which was 95% of the maximum strength from the average of the positive and negative loading cycles in the experiment.

Conclusions

The seismic behaviour of lightly reinforced concrete walls with openings subjected to damage by simulated seismic loading was investigated. The following observations and conclusions can be drawn.

- All three specimens eventually failed when the outermost reinforcing bars fractured and compressed concrete crushed and spalled severely near the foundation. Flexural cracks were concentrated near the bottom of the wall web and at the end of the cracks where they developed into inclined cracks. However, the flexural cracks of specimen W2 had a closer spacing and shorter developed length. Specimen W3, on the other hand, had cracks distributed throughout the wall web.

Figure 11. Specimen W2: (a) details of strut-and-tie model; (b) predicted theoretical strength (dimensions in mm)

- The test results of specimens W1 and W2 indicated that the arrangement of the openings did not have a significant effect on the behaviour of walls under reversed cyclic loading. The maximum strengths of W1 and W2 were similar, as were the stiffness degradations.
- Large openings in the wall web influenced the strength, ductility and secant stiffness of the wall. W3 had a significantly lower maximum strength of 350.4 kN, lower than that of W1 or W2. It can be concluded that the strength of an RC wall could be affected when the opening area on

Tie	Lumped reinforcement	Capacity: kN	f_u/f_y
AD	6T10	226.2	0.92
DH	6T10	226.2	1.00
DE	6T10	226.2	0.08
CF	6T10	226.2	0.70
FJ	6T10	286.7	0.99
FG	6T10	226.2	0.41
EK	6T10	226.2	0.32

Table 2. Main properties of ties of strut-and-tie model of specimen W2

Strut	Angle: degrees	Width: mm ^a	f_{cu}/f'_c
AC	30.4	445	0.20
CE	46.0	498	0.22
EG	52.5	257	0.47
GM	60.9	171	0.64
DF	45.0	495	0.01
FK	37.5	152	0.20

^a Strut width measured at the narrowest segment of the strut in Figure 11.

Table 3. Main properties of struts of strut-and-tie model of specimen W2

- the web is large. W1, W2 and W3 failed at ductility levels of 6, 5 and 7 respectively. W3 was the most ductile, with the largest lateral displacement. W3 had a slower stiffness degradation than specimens W1 and W2, which showed similar trends in stiffness degradation. The nine regular openings of W3 enabled a more ductile behaviour and lower stiffness degradation, which would be positive features in high-seismicity regions. W3 had a significant shear contribution of total lateral drift compared with W1 and W2. Specimens W1 and W2 showed a more pronounced flexural response.
- Strut-and-tie models for specimens W2 and W3 were developed using identified main load paths to form the force transfer mechanism for each specimen. The strut-and-tie model quantified the stress distribution in the main load paths and effectively explained the force transfer mechanism of each specimen. The predicted theoretical strengths of W2 and W3 using the strut-and-tie modelling approach were on the low side when compared with the experimental ultimate value.

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Figure 12. Strut-and-tie model for specimen W3 for predicting theoretical strength (dimensions in mm)

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