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SEISMIC DEFORMABILITY OF EXISTING RC BUILDINGS IN DHAKA BANGLADESH, THROUGH THE BASIC EXPERIMENT

A. Inoue [(1)], M. R. Islam [(2)], M. S. Rahman[(3)], N. Matsumura [(4)]

[(1)] *Senior Structural Engineer, OYO International Corporation (OIC), Tokyo Japan, inoue@oyointer.com*
[(2)] *Executive Engineer, Public Works Department (PWD), Government of the People's Republic of Bangladesh, rafiq89bd@gmail.com*
[(3)] *Executive Engineer, Public Works Department (PWD), Government of the People's Republic of Bangladesh, sohelr86@gmail.com*
[(4)] *Country Program Coordinator (Disaster Management), Japan International Cooperation Agency (JICA), Bangladesh Office, matsumuranaoki@gmail.com*

Abstract

Present Bangladesh National Building Code (BNBC 1993) became mandatory in 2006, and many existing RC buildings in Dhaka, the capital of Bangladesh, are not in compliance with the seismic requirements of the Code. Because of the low strength of concrete and small section sizes of a column, the numerical value of axial force ratio ($\eta = N/b{\cdot}D{\cdot}F_c$, N = axial force, $b{\cdot}D$= width and depth, Fc= compressive strength of concrete) is high. The shear reinforcement of a column is not enough. These conditions reduce the seismic deformability (ductility). An infilled brick wall in a RC frame is commonly used, which is a non-structural element, but it affects the behavior of a frame. An experimental approach was taken to get indicative information of the seismic deformability of existing RC buildings for the seismic assessment and retrofit.
The results indicate,

1. An impact of the axial force ratio on the deformability of a column is big. A frame with a column of the high axial force ratio (η=0.68) cannot expect the ductility, however a frame with an ordinary column (η=0.44) without an infilled brick wall has a reasonable ductility.
2. The increase of the stiffness and strength, and the decrease of the ductility are observed for a frame with an infilled brick wall. An infilled brick wall is a non-structural element, but it is not negligible for the assessment of a column.
3. The horizontal strength can be increased reasonably by introducing an infilled RC wall or a steel braced frame as a retrofit element, but the ductility and the vertical load supporting capacity cannot be expected after the maximum load in case the shear strength of boundary columns is low.

The strength evaluation by a conventional practical method is done to compare with the experimental result. Simplified monotonic load-deflection curves of specimens till the ultimate deflection that a column cannot support the axial load are prepared for a comprehensive understanding of the structural behavior. It is expected that the result of the experiment contributes for the proper seismic assessment and retrofit design of existing RC buildings in Dhaka Bangladesh and also in other developing countries. This experiment is the first one for a frame or a column in Bangladesh, and has been conducted as a part of the technical cooperation project (CNCRP) between PWD (Bangladesh) and JICA.

Keywords: Existing RC building; low strength concrete; axial force ratio; infilled brick wall; seismic deformability

1. Introduction

Existing RC buildings in Dhaka, the capital of Bangladesh have been said very vulnerable against earthquakes. The estimated number of existing RC buildings in Dhaka is more than 200,000. Present Bangladesh National Building Code (BNBC) 1993 became mandatory in 2006, and many existing RC buildings are not in compliance with the requirements of the Code. Evaluating the seismic deformability is the key together with the strength for a seismic assessment. According to the preliminary survey, a design concrete strength of concrete using brick-tips is 14N/mm2, and the value of axial force ratio of a column is approximately 0.50 at the ground floor for mid-rise existing RC buildings withof 20 to 30 year old. But the actual concrete strength is lower than the design value for not a few buildings, and the high axial force ratio is supposed. An infilled brick wall in an RC frame is commonly used, which is a non-structural element, but it affects the behavior of a column and a frame. Then an experimental approach was taken to get indicative information of the seismic deformability of existing RC buildings. Following factors which will affect the deformability of a column and a frame are incorporated in the experiment.

i) A column with the high axial force ratio using the low strength concrete and with the poor shear reinforcement, ii) A frame with and without the existance of an infilled brick wall, and iii) A frame retrofitted with an RC shear wall or a steel braced frame

2. Plan of Experiment

2.1 Experiment Model

An existing mid-rise RC frame, a scaled 1 span 1 story frame model has been selected for the structural experiment (Fig.1). The experiment was done in 2013 in addition to in 2012. To simulate the characteristics of existing RC buildings in Dhaka, specimens were prepared using brick aggregate concrete with low strength, 40 grade plain re-bar (yield stress= 275N/mm^2) with the detailing generally practiced during the construction period of 20 to 30 years ago.

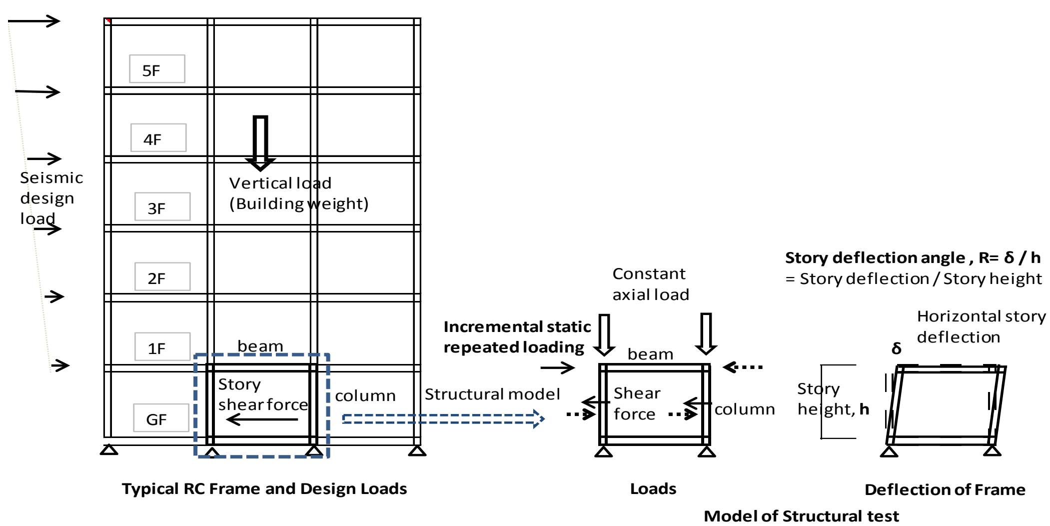


Fig. 1 - Experiment model

2.2 Experiment Apparatus

A load-meter and a dial gauge relation has been controlled for the loading. The data from a load-cell and a displacement transducer has been recorded by a data-logger. The constant vertical load (N=16 ton) and the repeated static incremental horizontal load were provided (Fig. 2). The loading program was 2 cycles each at the story deflection angle of 1/400, 1/200, 1/100, 1/50 and 1/25.

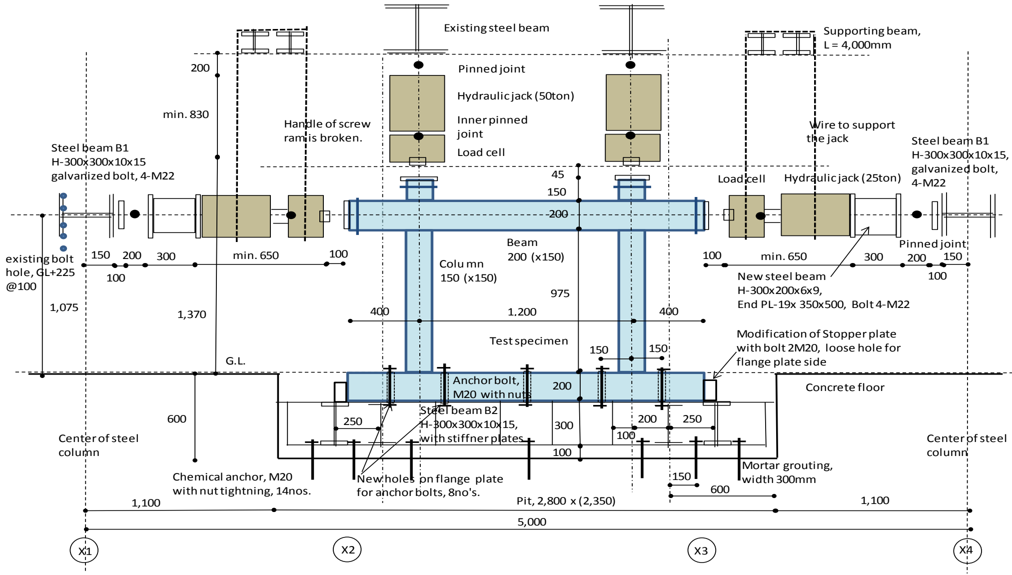


Fig. 2 - Experiment apparatus, unit: mm

2.3 Specimens

The specimens were planned considering existing frames not following the seismic requirements of BNBC 93 [1]. There are no clear regulations for the seismic evaluation of existing buildings in BNBC. The seismic evaluation and retrofit design based on the Japanese Standard and Guidelines [2] is planned to apply, but the low strength concrete (*fc*< 13.5N/mm2) is out of the scope work and infilled bricks wall is not the subject. The strict condition of the ductility is shown for an axial force ratio $N/b{\cdot}D{\cdot}F_c$ (N= axial force, $b{\cdot}D$= width and depth, and Fc= concrete strength) in the Japanese Standard. A report on the experimental study using the low strength concrete is referred [3]. Total 6 specimens including 2 retrofitted specimens were tested in 2013 (Fig. 3 and 4). Specimen No.1 is a standard specimen of a typical frame. The column size is 150mm ×150mm. The beam size is 150mm × 200mm. Plain bars were used. In specimen No.2, main re-bars of the beam has180 degree hooks. Specimen No.3 has a brick standing wall with thickness 65mm, and the height is 3/4 of the clear height of the column. A glass window was installed at the opening. Specimen No.4 has a brick infilled wall with no opening. Specimen No.5 is a retrofitted specimen by an infilled RC wall. Specimen No.6 is a retrofitted specimen by a steel braced frame. Because of the capacity limitation of horizontal hydraulic jacks, a retrofit with the low strength was planned.

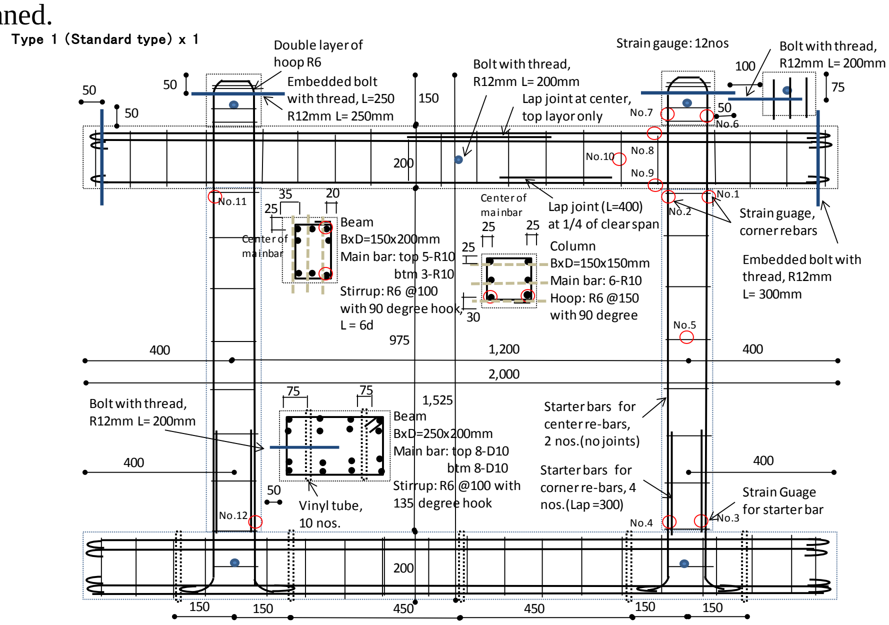


Figure 3 – Re-bar detail of experiment specimen No.1, unit: mm

Specimens were prepared with the concrete of low strength 10.6N/mm^2. The high axial force ratio ($N/b \cdot D \cdot F_c$= 0.68) is planned. Poor shear reinforcement is provided. The joint of plain bars is a lap joint.

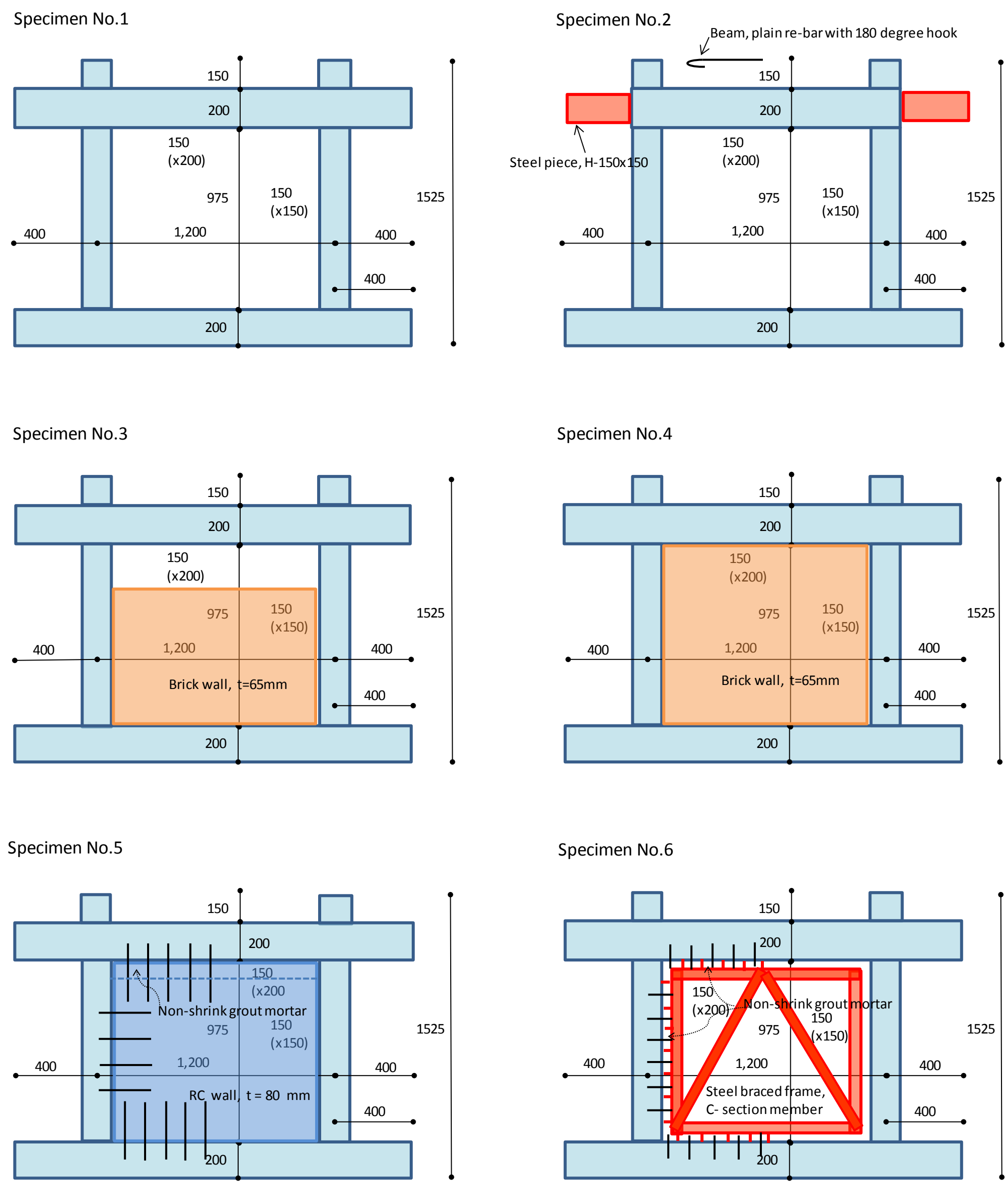


Fig. 4 - Experiment specimen, unit: mm

3. Results of Experiment

3.1 Limitation of the Experiment

The limitation of the experiment with respect to materials (Table 1) and the loading were observed as follows.

Material (Note: Low strength concrete is defined as the concrete strength less than 13.5 N/mm^2)

The concrete is low strength concrete 10.6 N/mm^2 at 8 weeks, compared to the concrete strength 16.5N/mm^2 in 2012.

The interval of shear reinforcement (column tie) was changed to @195 instead of @150, due to the use of re-bar φ7.45mm instead of planned φ 6.0mm to maintain the shear reinforcement ratio, through No.1 to No.6. The detail of steel bracing was also modified accordingly to meet the planned condition of the strength.

Table 1 – Limitation of materials

Material	Actual size	Actual yield stress	Original requirement	Yield stress in 2012
Plain re-bar	φ 10mm	350N/mm^2	φ 10mm, σ_y = 275N/mm^2	327 N/mm^2
Plain re-bar	φ 7.4mm	353N/mm^2	φ 6mm, σ_y =275N/mm^2	(560 N/mm^2)
Deformed re-bar	D10mm	274N/mm^2	D 10mm, σ_y = 275N/mm^2	387 N/mm^2
Steel angle plate	4.2mm	363 N/mm^2	3.0mm, σ_y =250N/mm^2	---

Loading

a. The slight inclination of the steel foundation beam under the specimen was observed in-plane and out-of-plane direction, and filler plates were provided where a gap exists under the specimen.
b. The capacity of horizontal hydraulic jacks is limited and was 23 ton (230kN) only.

3.2 Horizontal Load and Deflection Curve (Unit: metric ton and mm, 1 metric ton= 9.8kN)

The horizontal displacement was controlled by the reading value of a dial gauge at an upper side of the beam during loading. The horizontal load (by the load cell) and the deflection (by the displacement transducer) curves are shown in Fig.5. Failure patterns of each specimen are shown in Fig. 6.
(R= Story deflection angle (story drift ratio), $R= \delta / h$, δ = Horizontal deflection, h= Story height)

Specimen No. 1:
$R \leq 1/100$ ($\delta \leq$ 11.75mm), the max. load is 4.5 ton at positive loading and 4.0 ton at negative loading. Flexural cracks occurred at the top and bottom of columns. Vertical cracks also occurred at the bottom of columns. The diagonal cracks occurred at the top of column. $R \leq 1/50$ ($\delta \leq$ 23.5mm), the shear failure at the top of the right column occurred at positive loading 3.5 ton at 25.7mm, and the vertical load dropped.

Specimen No. 2:
$R \leq 1/100$ ($\delta \leq$ 11.75mm), max. load 3.9 ton is observed at positive and 5.0 ton at negative. Diagonal cracks extended at both sides of beam-column joint, and strength dropped from 3.5 ton (at 7.8mm) to 3.0 ton (12.4mm). Flexural cracks occurred at the bottom of columns. $R \leq 1/50$ ($\delta \leq$ 23.5mm), diagonal cracks extended at both panel zones. The cover concrete at the bottom of column (especially rear side) was detached. $R<1/25$ ($\delta \leq$ 47.0mm), the bottom of a column was failed. Cracks of panel zone were extended. The vertical load was reduced.

Specimen No. 3:
$R \leq 1/200$ ($\delta \leq$ 5.88mm), max. load at positive is 7.7 ton and negative is 6.0 ton. Cracks extended through columns and brick walls. $R \leq 1/100$ ($\delta \leq$ 11.75m), diagonal and vertical cracks extended at right column. Many flexural cracks at left column observed. $R \leq 1/50$ ($\delta \leq$ 23.5mm), positive load 7.0 ton dropped to 5.8 ton due to breaking of the glass. The shear failure of the right column occurred at 17.1mm at negative loading, and the vertical load also dropped.

Specimen No. 4:
$R \leq 1/200$ ($\delta \leq$ 5.88mm), max. load was 11.0 ton at positive and is 10.0 ton at negative. Diagonal cracks occurred at the top of left column. Cracks extended to the brick wall. $R \leq 1/100$ ($\delta \leq$ 11.75mm), the shear failure occurred at the left column at 10mm of positive loading. Diagonal cracks extended to the brick wall and extended through the bottom of right column. At negative loading, the horizontal stiffness decreased and could not support the vertical load.

Specimen No. 5:
$R \leq 1/200$ ($\delta \leq$ 5.88mm), max. load at positive is 23.3 ton (the limit of jack, at 4.3mm) and at negative is 21.2 ton. The horizontal cracks observed at the left column, the diagonal cracks developed at the wall. A small square hole was provided on the infilled RC wall of the specimen at the laboratory to reduce the strength, and loaded again. The shear failure occurred at positive loading 19.6 ton at 5.0mm. The axial load started to drop at around 8mm.

Specimen No. 6:
$R \leq 1/200$ ($\delta \leq$ 5.88mm), max. load is 22.0 ton at 5.2mm at positive, and is 22.0 ton at 6.2mm at negative. New diagonal cracks occurred at the middle of left column at the 3rd positive cycle. A slight buckling of the out-of-

plane direction of the left bracing was observed. R ≤1/100 ($\delta \leq$ 11.75mm), the shear failure occurred at the left column with 20 ton at 9.0mm at positive. The yield of the left side brace and out-of-plane direction buckling at the right side steel brace was observed. A horizontal crack beneath the upper RC beam at grout mortar was observed. The buckling of the vertical steel frame was observed. The drop of the vertical load started at 20.9mm.

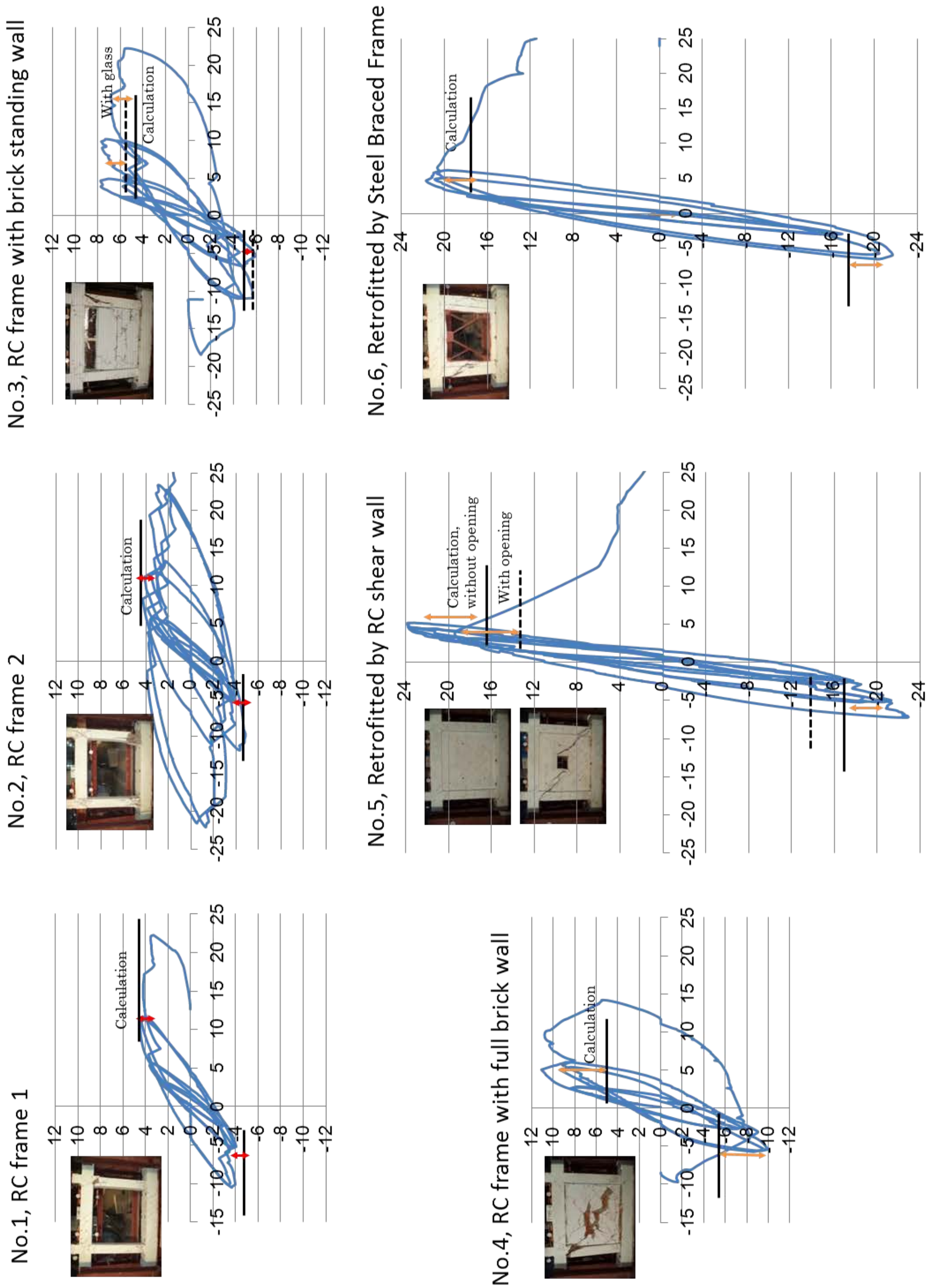


Fig. 5 - Horizontal load (metric ton) and deflection (mm) curve

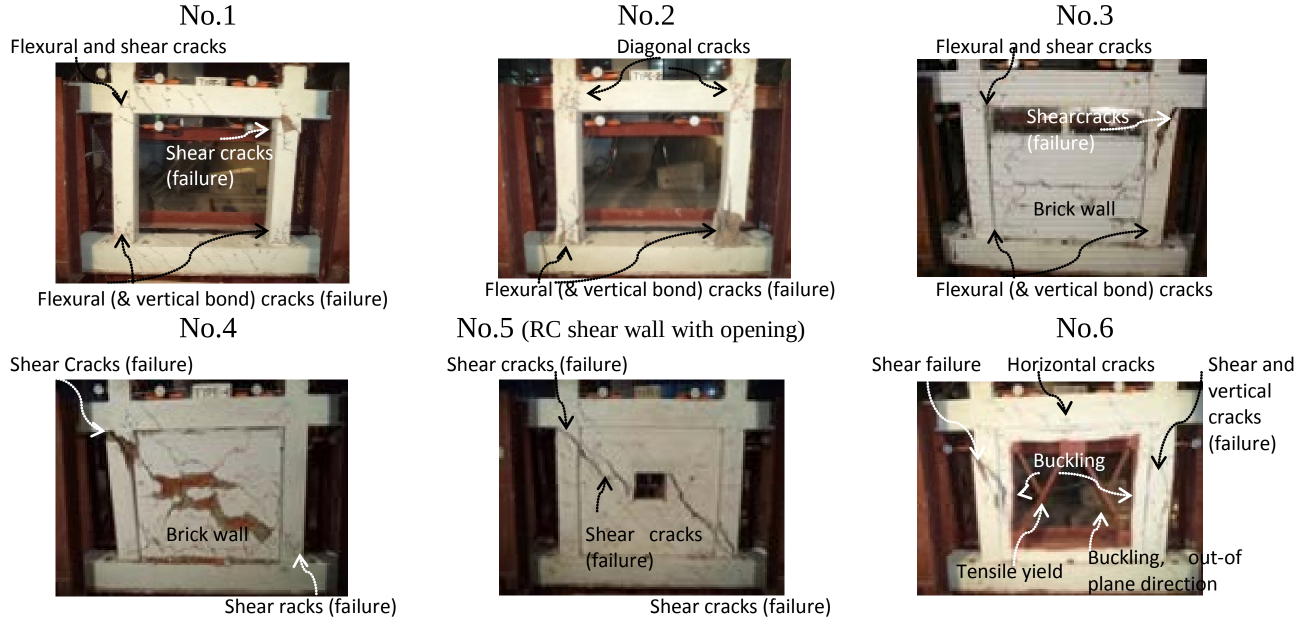


Fig. 6- Failure patterns of each specimen at the final stage

3.3 Initial Siffness

The observed initial stiffness of each specimen is, No.1: 1.3 ton/mm, No.2: 1.8 ton/mm, No.3: 6.6 ton/mm, No.4: 10.0 ton/mm, No.5: 25.0 ton/mm, No.6: 25.0 ton/mm. The initial stiffness of a frame with infilled brick walls of No.3 and No.4 is approximately 4 times and 6 times respectively that of average of No.1 and No.2. The initial stiffness of a frame with an RC wall (No. 5) and a steel braced frame (No. 6) is similar and is approximately 16 times that of average of No.1 and No.2.

4. Evaluation of Horizontal Strength (unit conversion; 1 metric ton= 9.8kN)

The horizontal strength of each specimen is calculated based on a conventional practical method. The calculated strength is summarized in Table 2 and is shown as horizontal lines in Fig. 5.

Specimen No.1: A supposed simple collapse mechanism of the frame is shown in Fig. 7. The flexural strength (M=10.5kN・m) of a column is calculated by the equation A1.1-1 of the Japanese Standard [2] (coefficient 0.7 was used instead of 0.8 of the Eq., because of the small dimension of a column). The shear force of two columns at the flexural strength is 43.2kN (Q=21.6kNx2). The shear strength (Q=30.8kN) of a column is calculated by the equation A1.1-2 (so called Arakawa's min. equation (0.053)) of the same Standard, and the reduction factor by the low strength concrete Kr= 0.83 [3] is applied for the calculation (Q=25.5kN). The calculation is slightly over estimated, and a reason will be a loss of the bond stress between plain bars and low strength concrete [3].

Specimen No.2: The calculated strength is same to that of specimen No.1. The calculated result is also slightly over estimated. Some drop of strength was supposed due to the anchor type of beam main re-bars and the shear failure of beam-column joint, but there was no such clear reduction of the strength in this specimen.

Specimen No.3: The left side column was evaluated as a short column due to the brick standing wall. The right side column is evaluated as flexural column similar to specimen No.1. The supposed clear height of column at left side is, $h_o= H_o- h_b/2$ (h_o= clear height of column, H_o= standard clear height of column, h_b= height of brick standing wall) as shown in Fig. 6. The shear force (Q=34.4kN, h_o= 0.61m) at the calculated flexural strength is bigger than the shear strength (Q=25.5kN), then the shear failure is supposed for the left side column. The horizontal strength of two columns were 47.1kN (=25.5kN+21.6kN). As far as the shear strength of a brick standing wall, clear information is not obtained from the experiment. Reference [4] introduces an average shear stress of brick wing wall at the strength as 0.20N/mm^2 as a seismic element, which is connected to a beam and a column for three directions. Similarly the average shear stress at the strength of brick standing wall, which is connected to a beam and a column for three directions, might be proposed in the range of 0.20~0.30N/mm^2 against brick wall section. This will be compared with shear strength of adjacent column whether shear failure of column occurs or not. This issue needs further investigation. The contribution of the strength of window with frame is not negligible, which is estimated roughly as 12kN.

Specimen No.4: The shear strength of columns (51.0kN= 25.5kNx 2) is estimated as the shear failure column, and the excessive strength is the contribution of brick walls. The average shear stress at the strength of infilled brick walls without opening is supposed in the range of 0.50~0.60N/mm^2 against brick wall section. Reference [5] introduces a failure and resisting mechanism as shown in Fig.8. Reference [4] introduces 0.60N/mm^2 as an average shear stress at the strength of an infilled brick wall as a seismic element. The contribution of infilled brick wall might be considered as a seismic element under the condition of no ductility of the frame.

Specimen No.5: The shear strength of post-installed anchor at a beam is designed so that this is higher than the shear strength of infilled RC panel. The shear strength is calculated as a column shear strength x 2 plus the shear strength of infilled RC panel. The direct shear failure [2] of a column was not observed in the experiment, since the shear strength of boundary columns is low. The total shear strength is estimated as 166.8kN (=115.8kN+25.5kNx2) without opening, and 135.4kN with opening. This strength evaluation of retrofitted specimen without opening and with opening of infilled RC wall panel for design purpose will be reasonable.

Specimen No.6: The shear strength of post-installed anchors and headed stud bolts are designed so that these are higher than the horizontal strength of steel brace. The direct shear failure of a column was not observed in the experiment. The Strength evaluation is the summation of strength of steel brace (tension brace 66.4kN and compression brace 58.4kN) and both columns (25.5kNx2). The total was 175.8kN. This strength evaluation for retrofit design purpose will be reasonable compared with the result of the experiment.

Table 2- Summary of the observed and calculated strength

Specimen No.	1	2	3	4	5	6
Observed max. strength (positive / negative), *H* (kN)	44.1/ 36.0	38.2/49.0	75.5/58.8	107.8/98.0	228.3(jack limit) /207.8 192.1(with opening)	215.6/215.6
(in metric ton)	4.5/ 4.0	3.9/5.0	7.7/6.0	11.0/10.0	23.3 (jack limit)/21.2 19.6(with opening)	22.0/ 22.0
Calculated strength, *Qu* (kN)	43.2 (21.6x2)	43.2 (21.6x2)	47.1 (25.5+21.6)	51.0 (25.5x2)	166.8 (115.8+25.5x2) 135.4 (with opening)	175.8 (66.4+58.4+25.5x2)
Supposed failure mode for calculation	Flexural col. x 2	Flexural col. x 2	Shear col. +flex. col.	Shear col. x2	Shear wall+ shear column x 2	Tensile and comp. braces+ shear col.x2
Calculated *Qu*/ observed *H*	0.98/ 1.20	1.13/ 0.88	0.62/ 0.80	0.47/ 0.52	0.73/ 0.82 0.70(with opening)	0.82/0.82
Observed initial stiffness (ton/mm)	1.3	1.8	6.6	10.0	25.0	25.0

Note: 1) The observed maximum strength in kN is derived from the observed value in metric ton multiplied by 9.8.

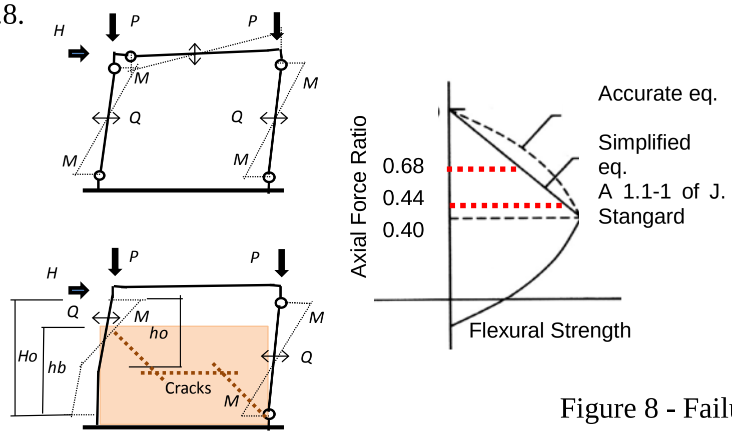


Figure 7 - Supposed collapse mechanism of a frame and M-N interaction curve of column

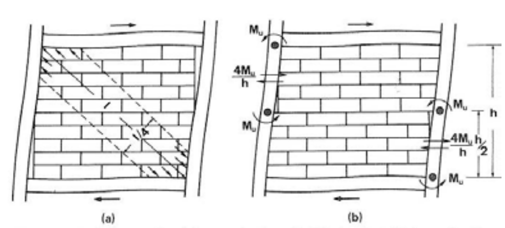


Figure 8 - Failure and resisting mechanism of infilled walls [5]

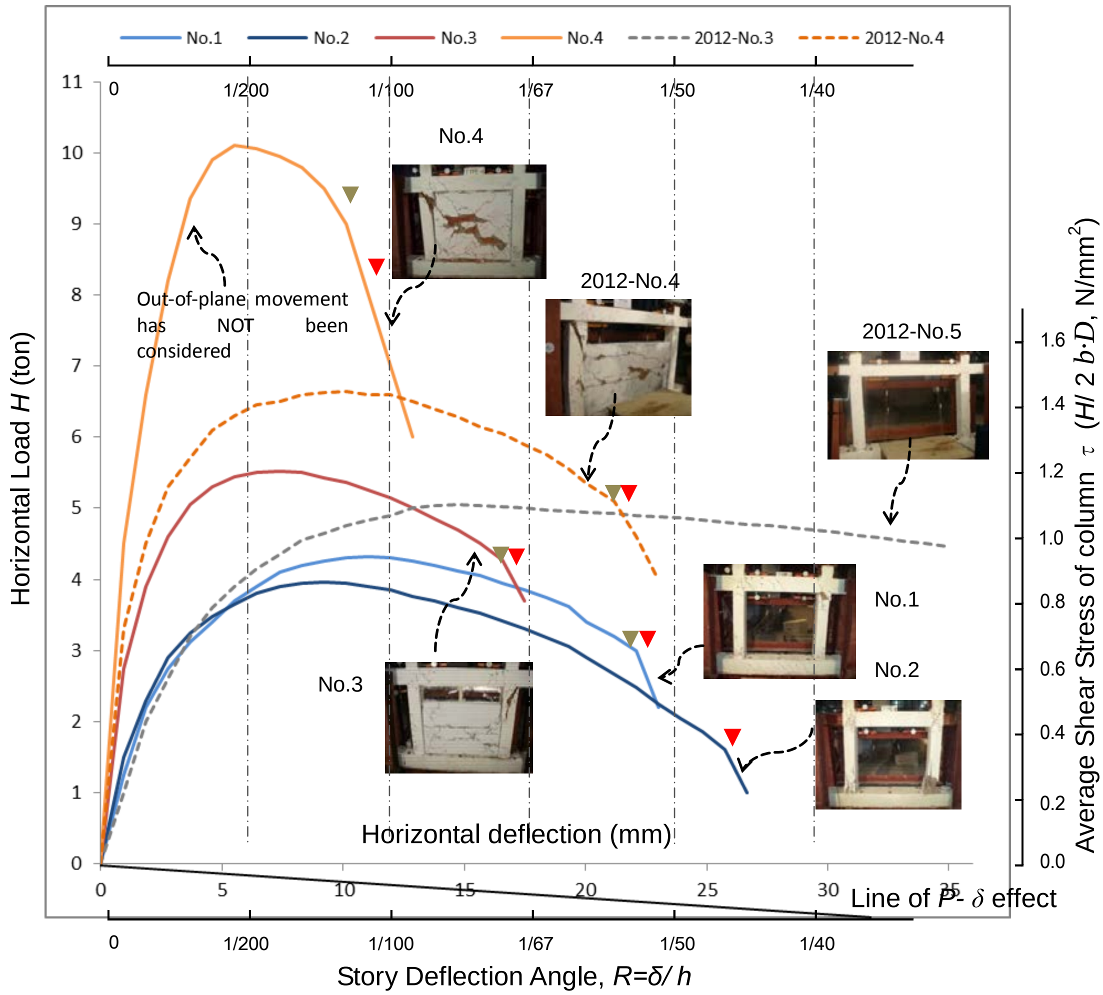
(a) Failure of a diagonal strut

(b) Horizontal sliding of a panel

5. Simplified Monotonic Horizontal Load- Deflection Curve

Simplified monotonic load-deflection curves (1/2) shown in Fig. 9 are drawn based on the result of the cyclic loading and the engineering judgment for the comprehensive understanding of structural behaviors. The deformation capacity of specimen No.1 (a standard specimen) and specimen No.2 (a failure of beam-column joint is supposed), of which the axial force ratio is very high (η =0.68), is around 1/100 expressed by the story deflection angle (story drift ratio). However specimen 2012-No.5, of which the axial force ratio is 0.44, has the deformation capacity more than 1/40. The initial stiffness and the maximum strength has been increased for specimen No.3, which has a brick standing wall, and the deformation capacity is around 1/200 at the maximum strength and more. However specimen 2012-No.4, with the ordinal concrete strength, has more deformation capacity up to around 1/100. The initial stiffness and the maximum strength has been increased for specimen No.4 which has an infilled brick wall without opening, and the story deflection angle at the maximum strength is around 1/200. The drop of the horizontal strength is very sharp after the maximum strength. The contribution of the brick wall as the average shear stress at the strength is supposed in the range of 0.50~0.60 N/mm^2.

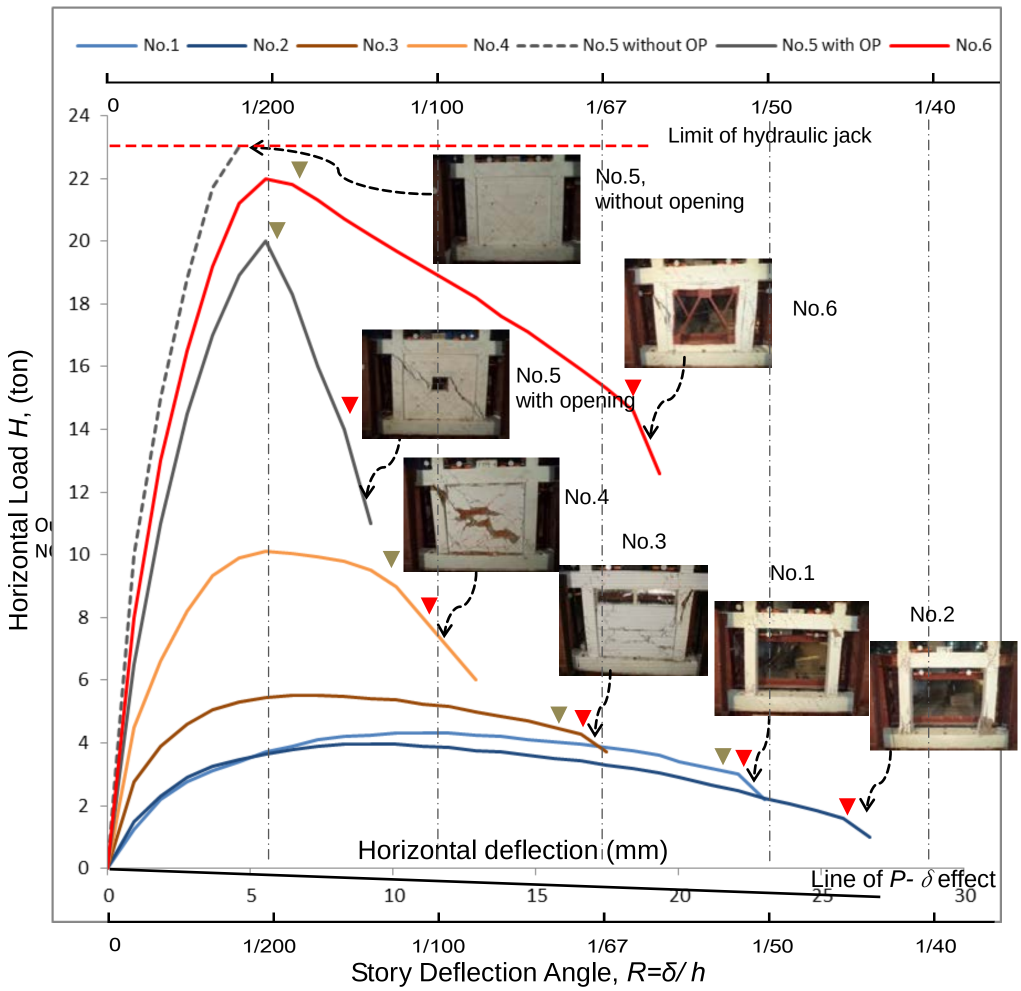
Axial force ratio: Specimen No.1 ~ No.4, $N/(b{\cdot}D{\cdot}F_c)$ = 0.68 (F_c=10.6N/mm^2, N=160kN)
Specimen 2012-No.4, 5, $N/(b{\cdot}D{\cdot}F_c)$ = 0.44 (F_c=16.5N/mm^2, N=160kN)



Note: Marking: ▼ denotes a point of "Drop in vertical strength".
▼ denotes a point of "Shear failure" by a visual observation.
R= Story deflection angle (story drift ratio) = Horizontal deflection (δ, mm)/ Story height (h=1,175mm)
$b \cdot D$= Width and depth of a column (mm× mm) F_c= Compressive strength of concrete (N/mm^2)
1 metric ton = 2, 205lbf = 9.8kN, 1Mpa = 1N/mm^2, 1N/mm^2 = 145 psi

Fig. 9 - Simplified monotonic load- deflection curves (1/2)

Simplified monotonic horizontal load- deflection curves (2/2) including retrofitted specimen No.5 and No.6 are shown in Fig. 10. Points showing the deformability, a story deflection angle at the maximum strength and st the drop in vertical strength are shown in the Figure. The horizontal strength of specimen No.5 and No.6 are controlled and are reduced to meet the capacity limit of the horizontal hydraulic jack. The horizontal strength can be increased reasonably by introducing an infilled RC wall (No. 5) or a steel braced frame (No.6) as a retrofit element. However the ductility and the vertical load supporting capacity cannot be expected after the maximum load. The horizontal load dropped sharply after the maximum load. This is caused because of the low shear strength of boundary columns due to the low strength concrete and the poor shear reinforcement. In case of a steel braced frame, the drop of the horizontal strength is not so sharp compared with that of RC wall. This will be caused by the contribution of posts of the steel braced frame.



Note: Marking: ▼ denotes a point of “Drop in vertical strength”.

▼ denotes a point of “Shear failure” by a visual observation.

R = Story deflection angle (story drift ratio) = Horizontal deflection (δ, mm)/ Story height (h=1,175mm)

$b \cdot D$ = Width and depth of a column (mm × mm), F_c = Compressive strength of concrete (N/mm^2)

Axial force ratio: Specimen No.1 ~ No.6, $N/b{\cdot}D{\cdot}F_c$ = 0.68 (F_c=10.6N/mm^2, N =160kN)

Fig. 10 - Simplified monotonic load- deflection curves (2/2)

6. Consideration

The concluding remarks are shown from the experiment with respect to the seismic deformability, the strength, and the retrofit.

Deformability: Generally the ductility of a column is evaluated based on the degree of the allowance against the shear failure of a column. The axial force ratio of a column is another important factor affecting the deformability. An RC column with the high axial force ratio (η=0.68) and without brick standing wall cannot expect the ductility (the story deflection angle (story drift ratio) at the ultimate capacity R_{mu}= nearly 1/100, ductility ratio $\mu \leq 1.0$), however an ordinary column (η=0.44) and without brick standing wall has reasonable ductility ($\mu \geq 3.0$). In case of a column with the high axial force ratio (η=0.68) and with brick standing wall, R_{mu} < 1/100 is supposed, however an ordinary column (η=0.44) and with brick standing wall, R_{mu} =nearly 1/100 is expected. A frame with brick walls without opening has little deformability (R_{mu}= nearly 1/200) in case the shear strength of boundary columns is low.

Strength evaluation: The flexural strength of a column and a frame are evaluated based on a conventional practical method. It will be requested to reduce the calculated flexural strength 10% to 20% in case of the combination of the low strength concrete and plain re-bars, because of the low bond strength. The effective column clear span (short column) is proposed in case of a column with a brick standing wall. It might be supposed that the average shear stress of a brick standing wall of 0.20~0.30N/mm^2 against brick wall section for the checking of the possibility of the shear failure of a column. In case of a frame with a brick wall without opening and the shear strength of a column is low, the shear strength of two columns is evaluated as the horizontal strength of a frame. If the shear strength of infilled brick wall is evaluated, the shear strength of infilled brick walls, which is estimated using the average shear stress of 0.50~0.60 N/mm^2, might be added under the condition of no ductility, and needs further study for retrofit design purpose.

Retrofit: There is a high possibility of the reduction of supposed seismic deformability in design for existing RC fames. This indicates that the strength and the stiffness oriented retrofit is practical. The ductility oriented retrofit allows a large deflection, and a beam-column joint and an infilled brick wall will be required to retrofit, but it will not be easy technically. The horizontal strength can be increased reasonably by introducing an infilled RC wall or a steel braced frame, but the ductility and the vertical load supporting capacity cannot be expected after the maximum load in case the shear strength of boundary columns is low. This is caused by the low strength concrete and the poor shear reinforcement, and a column jacketing will be required to expect the vertical load carrying capacity after the maximum load.

7. Conclusion

This basic structural experiment for a column/ a frame was done for the first time in Dhaka, Bangladesh to get indicative information for the assessment and retrofit of existing RC buildings, which are non-engineered buildings mainly. A few factors affecting the seismic deformability of RC buildings were studied through an experimental approach. The seismic deformability of an RC frame with a column of the high axial force ratio is very limited. Existing infilled brick walls increase the strength and decrease the deformability of a frame generally. The horizontal strength retrofitted by an RC infilled wall or a streel braced frame can be increased, but the deformability is limited in case that the shear strength of boundary columns is low. The story deflection angle at the maximum strength and at the drop in vertical strength is indicated as the useful information of the deformability of a column. The strength evaluation by a conventional practical method is done to compare with the experimental result. The result of the experiment has been incorporated for the modification of the method of Japanese Standard and Guidelines, and the development of technical manuals for existing RC buildings in Bangladesh [6], [7]. Since there are many issues to be solved, it is recommended to execute further experimental and analytical studies.

8. Acknowledgements

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