SEISMIC PERFORMANCE OF A RC BUILDING RETROFITTED AS A DUAL STRUCTURAL SYSTEM IN DHAKA, BANGLADESH

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Abstract

Introduction: A building plan has been prepared to extend the existing three-story non-engineered RC building, which is a garment factory, up to six stories in Dhaka the capital of Bangladesh. It has been required to retrofit and to extend the existing frame economically in a form satisfying the Bangladesh National Building Code (BNBC).
Seismic performance before retrofit: A detail building survey was conducted. Seismic evaluation was carried out by adopting the CNCRP manuals (JICA project), which is a Bangladeshi version of seismic evaluation and retrofit design method used in Japan, formulated according to the BNBC 2015 Draft. The seismic index of structure, *Is* value, of the ground floor was extremely low when the upper three stories were extended without retrofit of the lower stories, then the seismic retrofit was requested.
Seismic retrofit design: In order to improve the seismic performance, a six-story RC shear wall were properly installed in two directions, which is a dual system with moment resisting frame, to realize the extension of upper stories. The columns at lower three stories were reinforced with respect to the flexural and shear strength by RC jacketing. The foundation footings were strengthened to secure the supporting strength. In the lower stories, the seismic index of structure, *Is*, was calculated, and it was compared with the seismic demand index, *Iso*. The upper stories for the extension were designed in accordance with the BNBC.
Seismic performance after retrofit:
a) Pushover analysis: A pushover analysis was done to estimate the seismic performance. As a comparative study, the stiffness and horizontal strength was examined depending on the absence of the RC shear walls, and with and without the uplifting of the RC shear walls. By adopting this dual structural system, it has been confirmed that the plastic hinge formation is well balanced at each story, and it is possible to avoid the concentration of the deflection in a specific story. Possible ductility of each structural element was also studied.
b) Time-history response analysis: Degrading tri-linear restoring force characteristic was set at each story based on the pushover analysis. Assuming a shear type response model, artificial seismic waves corresponding to the elastic acceleration response spectrum of the BNBC were inputted, and the performance was estimated through the response results.
Retrofit construction: A uniting work to the existing columns on both sides for the new RC shear walls and reinforcing bar/ concrete work, RC jacketing work on existing columns, strengthening work of foundation footings were provided.
Conclusion: The existing three-story RC building was effectively extended vertically and seismically upgraded by the adoption of a dual structural system using six-story RC shear wall. A pushover analysis and a time-history response analysis were conducted to verify the earthquake resistance capacity, and the effectiveness of this system has been shown. Further issues on the discussion would be the desirable retrofit design of multi-story RC shear wall and moment frame including existing beams. This study was conducted as a part of the technical cooperation projects (CNCRP and BSPP) between Public Works Department (PWD) of Bangladesh and Japan International Cooperation Agency (JICA).

Keywords: Seismic retrofit; RC building; Dual structural system; Ductility; Dhaka

1. Introduction

A plan has been prepared to extend the existing three-story non-engineered RC building, which is a garment factory, up to six stories in Dhaka the capital of Bangladesh. It has been required to retrofit economically and to extend the existing frame up to six stories in a form satisfying the Bangladesh National Building Code 2015 Draft [1]. However, there was no detail description for the seismic retrofit design in the BNBC.

Fig. 1 - Existed building (left) and the building after retrofit (center and right)   Source: JICA project

2. Seismic performance before retrofit

A detail building survey was conducted. Seismic evaluation and retrofit design were carried out by adopting the CNCRP manuals developed by the JICA project [2], which is a Bangladeshi version of seismic evaluation and retrofit design method used in Japan [3], and were formulated according to the requirements of the BNBC. The Seismic Index of Structure "*Is*" of the ground floor was extremely low when the upper three stories were extended without retrofit as shown in Fig. 6 (left), then the seismic retrofit was requested.

3. Seismic retrofit design

In order to improve the seismic performance, a six-story RC shear wall were properly installed in two directions, which becomes a dual structural system, to realize the extension of the upper stories. Refer to Fig. 2. The columns at lower three stories were strengthened for the flexural and shear strength by the RC jacketing. The foundation footings were reinforced to secure the supporting strength. A repair work was done to improve the durability, but the seismic retrofit work was not planned for existing beams because of the difficulties of the work. The upper stories for the extension were designed in accordance with the BNBC.

3.1 General information

The estimated seismic weight of the building is shown in Table 1.

Table 1 – Seismic weight of the building   Source: JICA project

Level	Wi (kN)	Σ Wi (kN)
PH	434	434
6	6,585	7,019
5	8,556	15,574
4	8,556	24,130
3	8,556	32,685
2	8,556	41,241
1	8,785	50,026

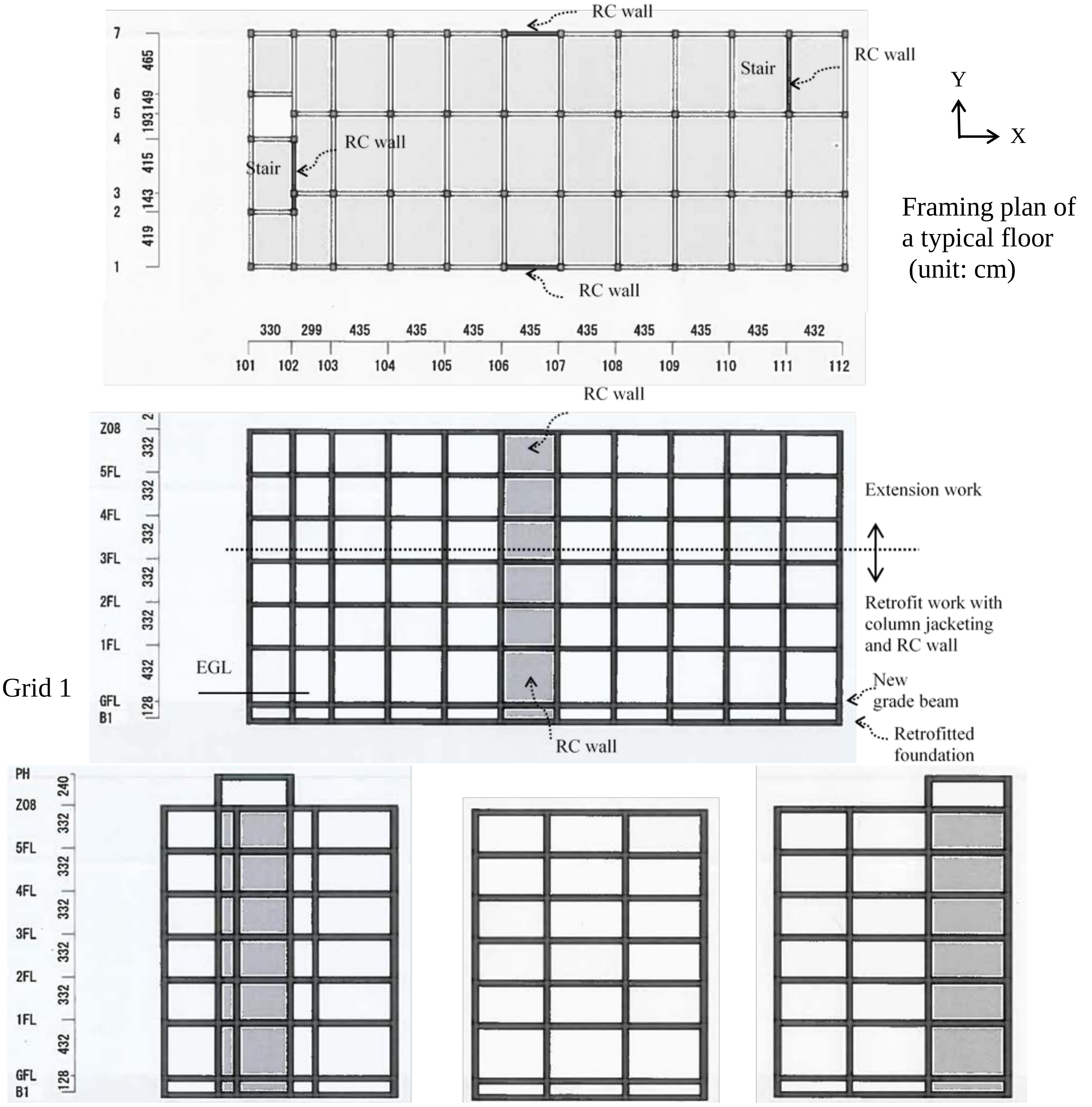
Live load, typical: 3,000N/m^2 for frame
2,000N/m^2 for seismic load
roof: 1,500N/m^2 for frame
1,000N/m^2 for seismic
Floor finish, typical: floor 1,430N/m^2+ 1,190N/m^2 for wall
roof: 2,000N/m^2

Used materials, Concrete: Lower stories, existing beam fc=10.8N/mm^2, column fc=20.3N/mm^2 (weighted average of existing and new), wall fc=25N/mm^2, floor fc= 10.8N/mm^2, grade beam fc=25N/mm^2. Upper stories (extension), fc=25N/mm^2, Re-bar: existing fb=275N/mm^2, Retrofit and extension fb=400N/mm^2.

Typical internal columns with the sizes of 300mm x 300mm were retrofitted to the sizes of 500mm x 500mm.

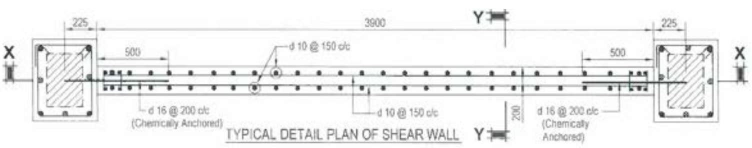
3.2 Framing plan and elevation

A framing plan and framing elevations are shown in Fig. 2. Typical section of a RC wall for X direction and a column jacketing is shown in Fig. 3.



Grid 102 Grid 103~Grid109 Grid 111

Fig. 2 - Framing plan and elevation (unit: cm) Source: JICA project



Column: B x D = 450 x 575mm
Londitudinal bar: 4-D25 + 4-D16
Transverse bar: D10 @100

Fig. 3 - Typical section of a RC wall and a column jacketing for the retrofit Source: JICA project

3.3 Results

The seismic index of structure, *Is*, was calculated based on the 2nd level screening method, which supposes the collapse of vertical structural elements. Relation of Strength index *C* and Ductility Index *F* before (left, in case vertical extension is done without retrofit) and after the retrofit (right) at the ground story is shown in Fig. 6. Proposed Seismic Demand Index, *Iso*, is expressed by the following equation, which is 80% of the design elastic acceleration response [2]. *Cs* (Normalized acceleration response spectrum) is shown in Fig.5.

$$I_{so} = 0.8 \times \frac{2}{3} \times Z \cdot I \cdot C_S \qquad \text{(1.1) of [2]} \qquad (1)$$

Where,
I : Structure importance factor, *I*= 1.0
Cs : Normalized acceleration response spectrum, *Cs*=2.25 for soil type *SC* (stiff soil)
Z : Seismic zone coefficient, *Z*= 0.2, as shown in Fig. 4

Zone 4: *Z*= 0.36
Zone 3: *Z*= 0.28
Zone 2: *Z*= 0.20
Zone 1: *Z*= 0.12

Fig. 4 - Seismic Zoning Map

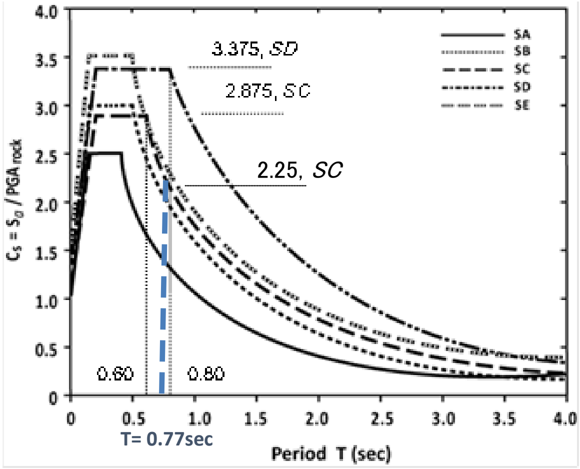


Fig. 5 - Normalized acceleration response spectrum, *Cs*

Hence, *Iso*= 0.8x2/3x 0.2x1.0x2.25= 0.24 was applied. Calculated Basic Seismic Index of Structure *Eo* = 0.159 (Strength Index, *C*) x 2.6 (Ductility Index, *F* of frames) = 0.41, then *Is*= 0.41 for X direction as shown in Fig. 6 (right), where S_D (Irregularity Index) = 1.0, and *T* (Time Index) = 1.0. Ductility Index of six-story RC shear wall was evaluated as 2.0. As for Y direction, *C*= 0.179 and *F*= 2.60, then *Eo*= 0.467, then *Is*= 0.467. In case *Cs* = 2.875 (max. value), then *Iso* = 0.30 for information.

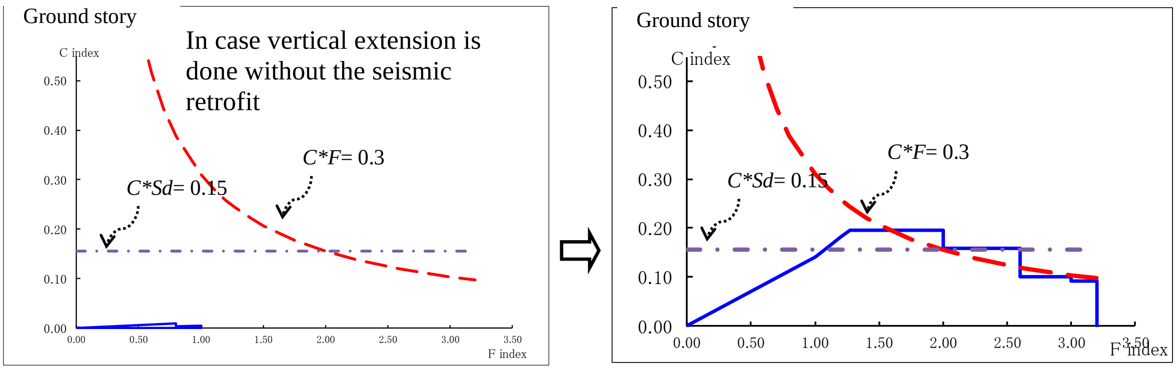


Fig. 6 - Relation of Strength Index *C* and Ductility Index *F* based on 2nd level screening method before (left) and after retrofit (right) for X direction at the ground story Source: JICA project

The design base shear force of a dual structural system by the BNBC was calculated as follows. Dual shear wall and frame system: ordinary RC moment frame (OMF) and ordinary RC shear wall, *R*=4.5 was applied. Base shear coefficient, *V*/*W*=2/3**Z***I***Cs*/*R*=2/3x 0.20x 1.0x 2.25/ 4.5= 0.0667. Building height= 22.5 m, *T*= 0.77 sec. Soil type *SC* (Stiff soil), *Cs*=2.25 < 2.875, then, *V* =0.0667x 50,026 (Building weight) = 3,335kN

4. Seismic performance after retrofit

The existing structure have been retrofitted by introducing six-story RC shear walls and RC jacketing on columns. But these are vertical structural elements and the consideration of beam collapse mode was required. Then, a push over analysis and a time-history response analysis were conducted to assess the seismic performance of the building after the retrofit.

4.1 Pushover analysis

A pushover analysis was done to evaluate the seismic performance of the building. A basic idea of 3rd level screening method of Japanese Standard [3] was utilized for the evaluation of the ductility of frames. As a comparative study, the stiffness and the horizontal strength was examined under the condition of the absence of the six-story RC shear wall, and with and without uplifting of the six-story RC shar wall. By adopting this dual structural system, it was confirmed that the plastic hinge formation was well developed at each story, and it is effective to avoid the concentration of the deflection in a specific story.

4.1.1. Conditions and results

A non-linear static behavior of a dual structural system was evaluated by a pushover analysis. The criteria of judgement for the brittle failure of structural elements was set as shown in Table 2.

Table 2 - Criteria of judgement for the brittle failure of structural elements   Source: JICA project

	Beam	Column	RC wall
Shear failure	Excluding the member (no horizontal strength) and continue the calculation. (limit of ductility factor of the frame is presumed as 1.7 at lower stories.)	Stop the analysis.	Check the point of shear failure, and continue the calculation maintaining the strength, which is reference only. 1.1 (premium rate) x Q (shear force) > Qu (shear strength), is applied for the judgement of shear failure.
Axial failure	-----	Maintaining the strength and continue.	Maintaining the strength and continue the calculation.

The results are summarized as follows and are shown in Fig. 7 and Fig. 8.

a) X direction:

・No shear failure of columns was observed, and beam collapse mode was shown for the frames.
・Flexural failure mode was observed for the six-story RC shear wall.
・Shear failure of existing some beams was observed at the range of maximum strength at lower stories.
・Horizontal load carrying capacity at the ground story was estimated approximately 7,700kN at the story deflection angle (= story deflection/ story height) with 1/100 rad. C=7,700/50,026=0.154. RC shear wall portion was estimated as 3,200kN out of 7,700kN. The over-strength factor, Ωo=7,700/3,335= 2.31

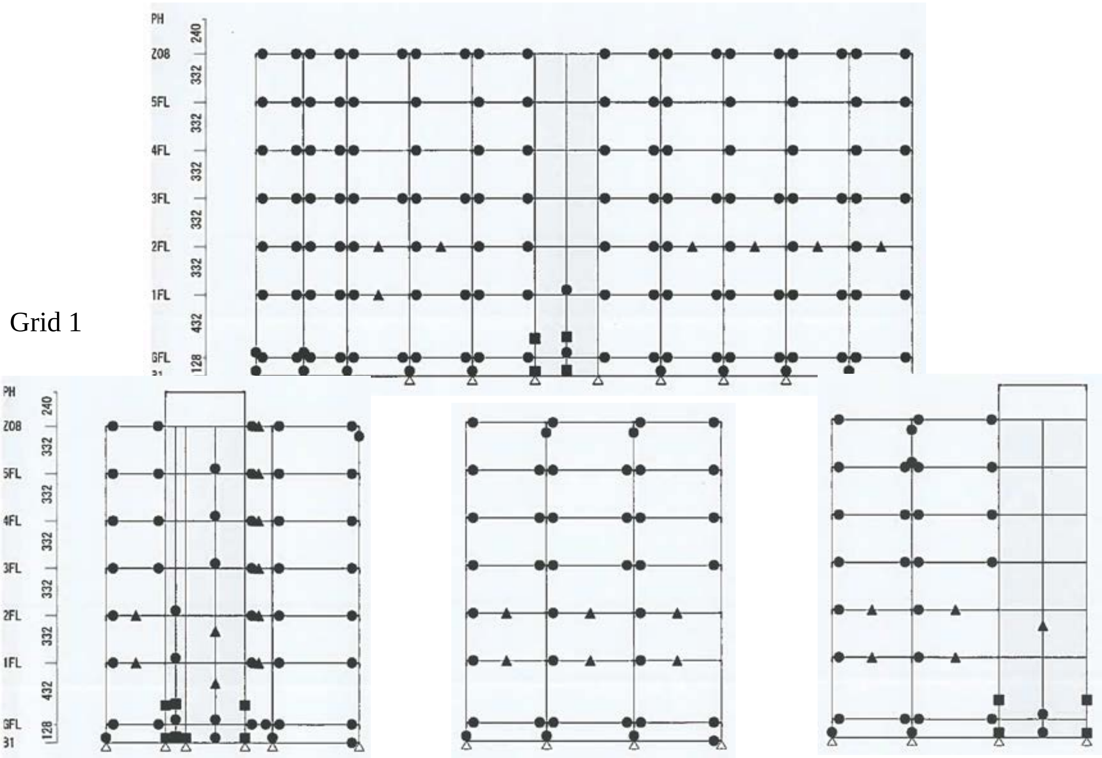
b) Y direction:

・No shear failure of columns was observed, and beam collapse mode was shown for the frames.
・Flexural type deflection was observed at first for RC walls but shear failure was observed at the later stage.
・Shear failure of existing some beams was observed at the range of maximum strength at lower stories.
・Horizontal load carrying capacity at the ground story was estimated approximately 11,500kN at the story deflection angle with 1/100 rad., C=11,500/ 50,026= 0.230. RC shear wall portion was estimated as 5,900kN out of 11,500kN. The over-strength factor, Ωo=11,500/3,335= 3.45

In order to evaluate the ductility of moment frame with shear failure of exisitng beams, following process was done. The ratio of “shear strength/ shear force at flexural yield” was evaluated, and the Ductility Index bF for beams was in the range of 1.5 to 2.2 based on Eq. (26) [3]. Ductility Index of six-story RC shear wall was estimated as 2.0. Then the average ductility index F was presumed as 1.90 for X direction, which indicates the ductility factor μ = Rmu/ Ry= 1.70. The relation of F and μ is shown in the following equation.

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \qquad F \leq 3.2, \text{ Eq. of (16) [3]} \qquad (2)$$

Grid 1

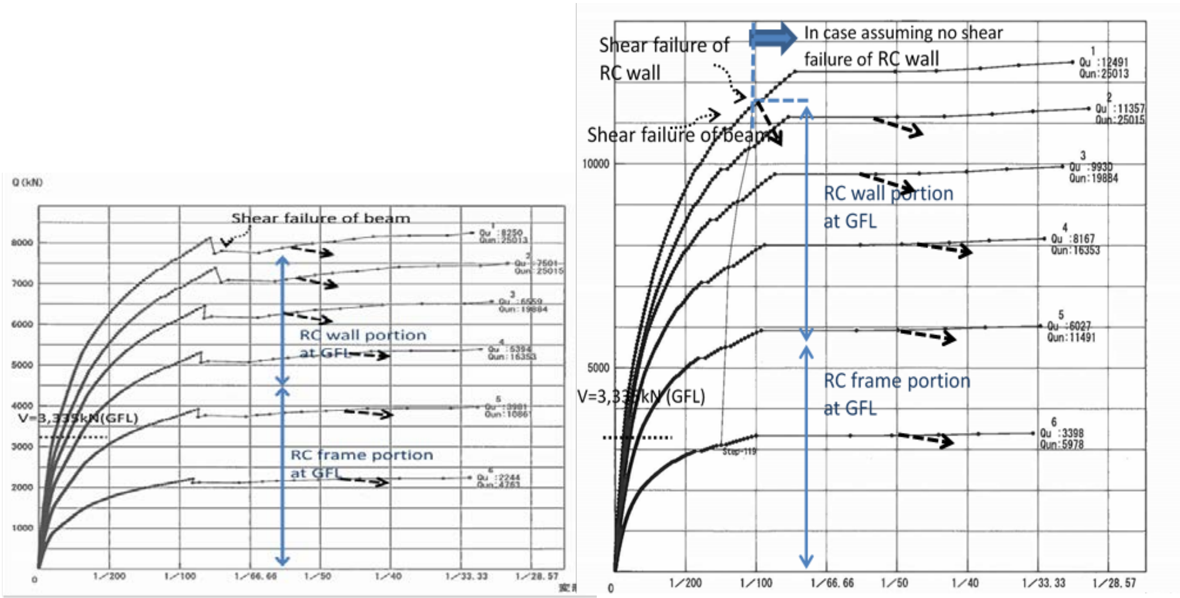


Grid 102　　Grid 103~ grid 109　　Grid 111

Symbol: ●: Plastic hinge ▲: Shear failure ■: Axial failure

Fig. 7 - Failure pattern (Plastic hinge formation and Collapse mechanism, unit: cm) Source: JICA project

The relation of the story shear force and the horizontal deflection angle (story drift ratio) is shown in Fig. 8.



X direction　　Y direction

Fig. 8 - Relation of story shear force and story deflection angle (story drift ratio)　Source: JICA project

"*Is*" at the ground story was estimated as, *Eo*= *C* x *F*= 0.154 x 1.90= 0.292, then *Is* = 0.292 > *Iso* = 0.24 for X direction. *Eo* = *C* x *F*= 0.230 x 1.0= 0.230, then *Is* = 0.230 < *Iso* = 0.24 for Y direction. The result of Y direction is not satisfying the criteria slightly, and the main reason of the shear failure of RC shear wall would be that 1.1x *Q* was applied for the judgement and the effect of boundary beams. Then a time-history response analysis was conducted as a next step as shown in Section 4.2.

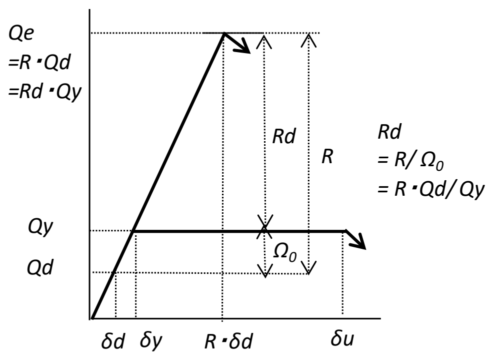
4.1.2 Evaluation of Response reduction factor *R*

Response reduction factor *R*= 4.5 was applied for this dual structural system in the BNBC. The over-strength factor Ω and ductility factor μ was calculated by a pushover analysis to examine the adopted *R* value. An idea of *Rd* (modified *R*), which is shown in Fig. 9, is introduced and was assumed to be like a Ductility Index *F* of Japanese Standard [3]. The value of over-strength factor Ω_0 multiplied by *Rd* (modified *R*) is 4.39 for X direction and 4.38 for Y direction respectively as shown in Table 3 for information. Calculated values are very close to applied *R*= 4.5 as OMF of a dual structural system.

Table 3 - Estimation of Response reduction factor *R* at the ground story Source: JICA project

X direction					Y direction				
Ω_0	R_{mu}	*Rmu/Ry*	*F* (=*Rd*)	*R*= Ω_0 x *Rd*	Ω_0	R_{mu}	*Rmu/Ry*	*F* (=*Rd*)	*R*= Ω_0 x *Rd*
2.31	1/53 rad. (δu=82mm)	1.70	1.90	2.31x1.90= 4.39 ≤ 4.5	3.45	1/100 rad. (δu=44mm)	1.0	1.27	3.39x1.27= 4.38 ≤ 4.5

Where:
Rmu: Story deflection angle (story drift ratio) at the ultimate capacity, = $\delta u/h$ (story height=4,320mm)
Ry: Story deflection angle at the yield, 1/100 rad. was supposed by the pushover analysis in this study.
F: Ductility Index of the Japanese Standard, eq. (16)
Ω_0 : Over-strength factor
Rd: Modified *R* and is supposed to be like *F* of the Japanese Standard.
Some notations are explained more using Bi-linear model as shown in Fig. 9.



Where;
Ω_0 : Over-strength factor = *Qy/Qd*
Qd : Design base shear force ("*V*" is used instead of "*Q*" in the BNBC)
Qe : Elastic design response base shear force= *R* ・*Qd*
Qy : Yield strength or horizontal load carrying capacity, *Qu*
R : Response reduction factor and 4.5 for OMF of a dual system
Rd : Modified response reduction factor, and is expressed by *R* divided by over-strength factor Ω_0.
δu : Horizontal deflection at the ultimate stage
δy : Horizontal deflection at the yield

Fig. 9 - Explanatory figure of modified response reduction factor Source: JICA project

4.1.3 Load deflection curve of the RC moment frame in case there are no RC shear walls

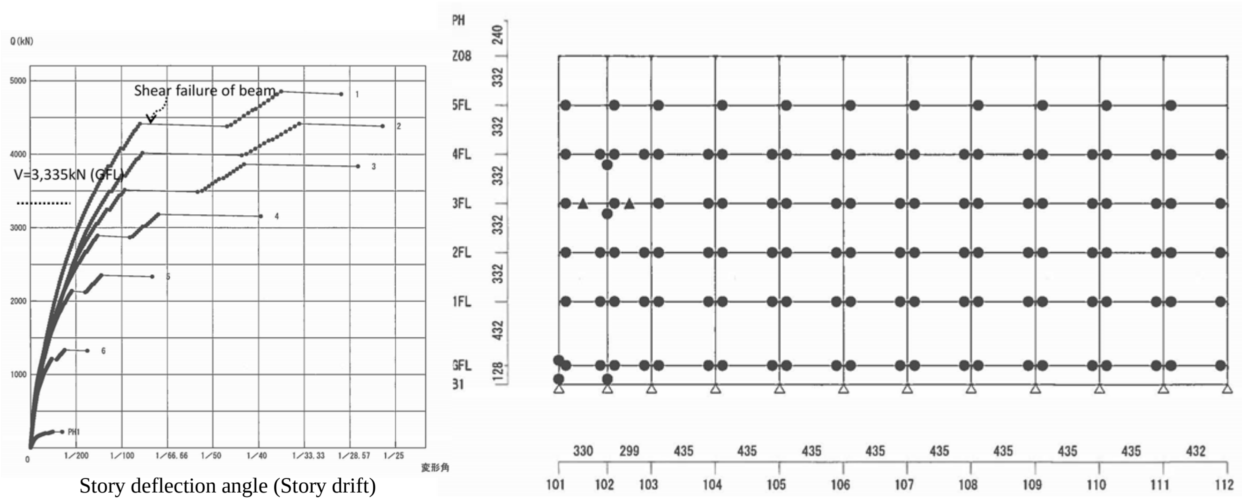
A pushover analysis of the moment frame model without six-story RC shear walls was conducted for the comparison purpose. The result is summarized as follows.

a) X direction
・No shear failure of columns was observed, and beam collapse mode was shown for frames.
・Shear failure of existing beams at some elements was observed at the range of maximum strength.
・Horizontal load carrying capacity was estimated approximately 4,400kN at the deflection angle with more than 1/100 rad., (*C*= *V*/*W*= 4,400/50,026= 0.088).

b) Y direction

- No shear failure of columns was observed, and beam collapse mode was shown for frames.
- Shear failure of existing beams at some elements was observed at the range of maximum strength.
- Horizontal load carrying capacity was estimated approximately 5,400kN at the deflection angle with 1/100 rad., (C= V/W=5,400/50,026= 0.11)

Plastic hinges at upper stories were not developed as shown in Fig. 10 b). The effect of stress redistribution by the six-story RC shear wall was expected by the comparison with Fig. 7. In addition, the six-story RC shear wall seems contribute to the optimization of the horizontal strength distribution at the upper and lower stories by transferring the shear force at a certain story which caused the loss of strength of some beams.



a) Story shear force and story deflection angle (Story drift ratio, X direction)

b) Formation of plastic hinges (grid 1, X direction)

Fig. 10 - Behavior of frames in case there are no RC walls Source: JICA project

4.1.4 Load deflection curve of RC moment frame with RC shear wall incorporating the uplift

Load deflection curve of RC moment frame with six-story RC shear wall incorporating the uplift of the foundation was studied for reference only.

a) X-direction

- Uplift of RC wall for tension side was observed, and the shear failure of RC shear wall was not observed.
- Horizontal load carrying capacity was estimated approximately 7,200kN (C=7,200/50,026=0.144)
- Strength of RC shear wall was reduced from 3,500kN to 2,700kN by the effect of uplifting but more deformation capacity was expected.
- Overs-strength factor, Ωo=7,200/3,335= 2.16

b) Y-direction

- Uplift of RC wall for tension side was observed, and the shear failure of RC shear wall was not observed.
- Horizontal load carrying capacity was estimated approximately 10,500kN (C=10,500/50,026=0.210).
- Strength of RC shear wall was reduced from 6,500kN to 5,000kN by the effect of uplifting but more deformation capacity was expected.
- Overs-strength factor, Ωo=10,500/3,335= 3.15

4.2 Time-history response analysis

Degrading tri-linear restoring force characteristics were set at each story based on a push-over analysis. Assuming a shear type response model, artificial seismic waves corresponding to the elastic acceleration response spectrum of the BNBC were inputted, and the performance was assessed through the response results.

4.2.1. Analysis conditions

Supposed vibration model: Shear type six lumped mass model

Restoring force characteristics: Degrading Tri-linear model based on the result of push-over analysis was used as shown in Fig. 11.

Damping constant, h=3.0% stiffness proportional type was supposed, tangential stiffness was not considered.

4.2.2. Input earthquake waves

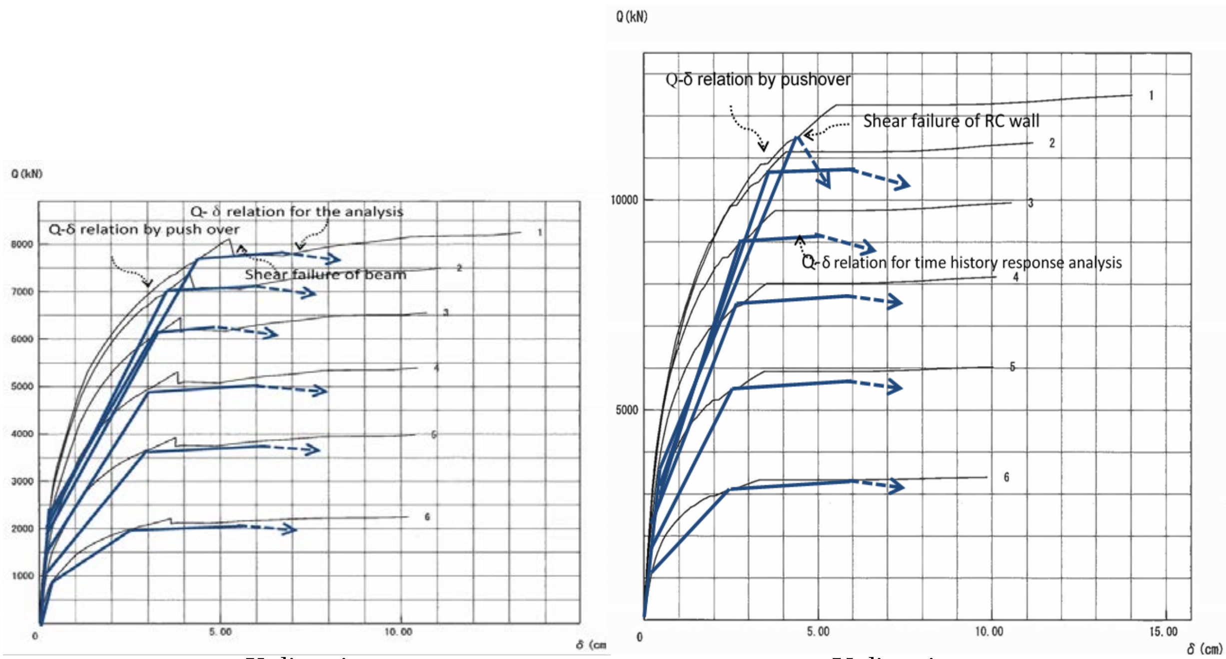
Waves; Three artificial waves complying the elastic response spectrum of soil type *SC* (Stiff soil) of the BNBC [2]. Inputted PGA is equivalent to 0.153G (2/3x 0.20G x 1.15 (soil factor for dynamic analysis)). The peak value of the elastic response is 0.440 (0.383x 1.15).

4.2.3. Proposed acceptance criteria

X direction: Response ductility factor is less than 1.7. (Story deflection angle: 1/58 rad., ultimate state of existing beam elements)

Y direction: Response ductility factor is less than 1. (Story deflection angle: 1/100 rad., shear failure state of RC wall elements)

Due to the limitation of the response analysis software, it was not possible to evaluate a decrease in strength (negative slope). It is known that the deflection proceeds rapidly in the negative slope region.



X direction　　Y direction

Fig. 11 - Supposed restoring force characteristics (Tri-linear model)　Source: JICA project

4.2.4 Results

a) X direction:
Building period $T1$= 0.73sec. Response ductility factor was less than 1.0 at each story. Story deflection angle was less than 1/200 rad., which satisfied the proposed acceptance criteria, as shown in Fig. 12. The average of response base shear force was 4,136N (= 0.0826x W) and was 1.24 times of design base shear force, which is V= 3,335kN.

b) Y direction:

Building period *T*1= 0.58sec. Response ductility factor was less than 1.0 at each story. Story deflection angle was less than 1/100 rad. (max. was 1/259 rad. at 1st story (level 2)), which satisfied the proposed acceptance criteria. The average of the response base shear force was 5,537kN, and was 1.66 times of design base shear force, which is *V*= 3,335kN.

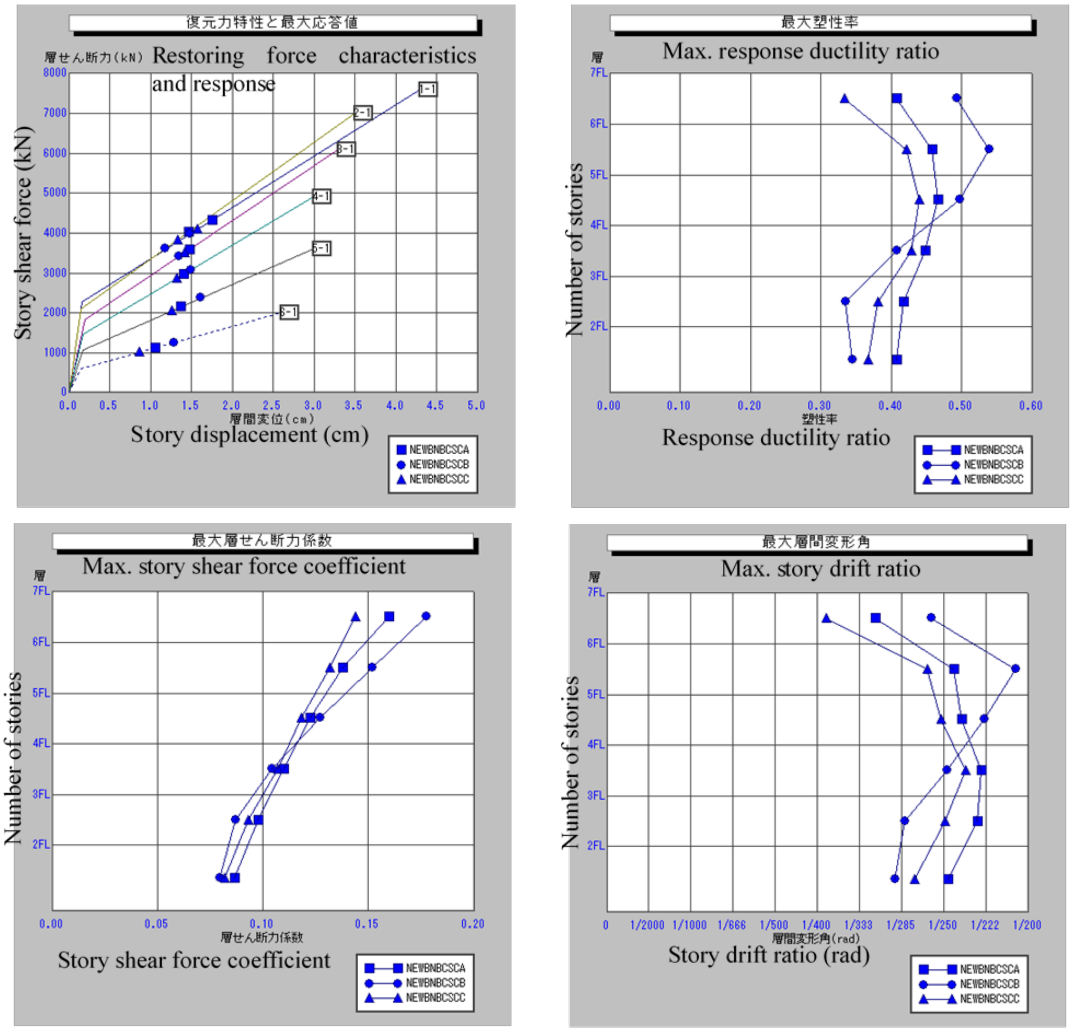
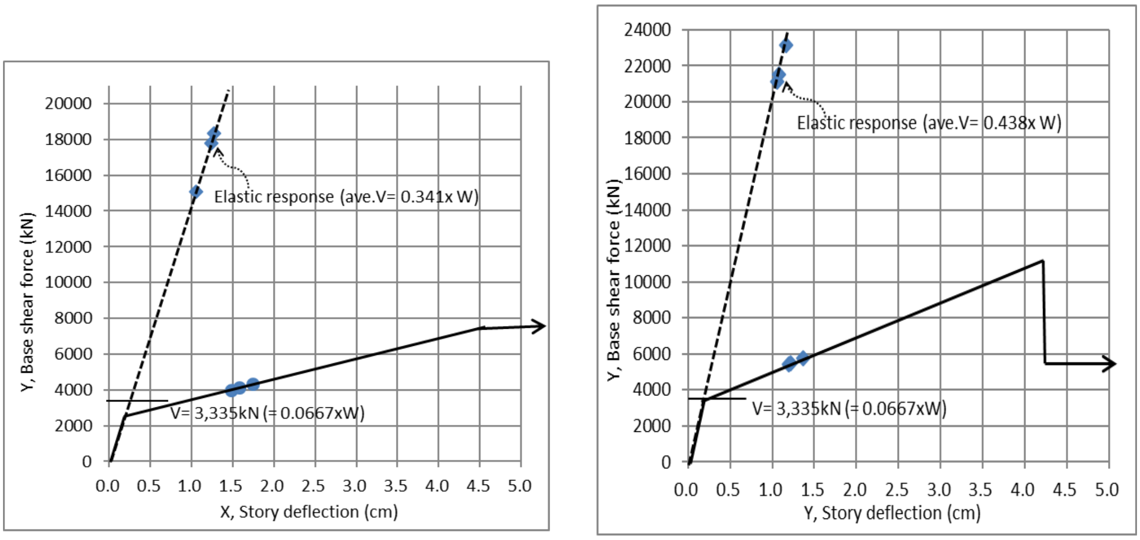


Fig. 12 - Results of response analysis for X direction   Source: JICA project

c) Comparison with the elastic response and additional analysis

The response results in two directions at the ground story are shown in Fig. 13. The linear elastic response value (damping constant 3%) based on the initial elastic stiffness was also shown in the same Fig. for comparison purpose. Elastic shear coefficient was 0.341, and was 78% of the peak value, which is 0.440 (0.383x 1.15) for X direction. Elastic shear coefficient was 0.438, which is 100% of the peak value (0.383x 1.15) for Y direction.

Regarding the response deflection, average of the non-linear response value divided by the average of the linear response deflection was 1.34 for X direction and 1.15 for Y direction respectively. The result shows it is close to so called “Constant displacement principle”.

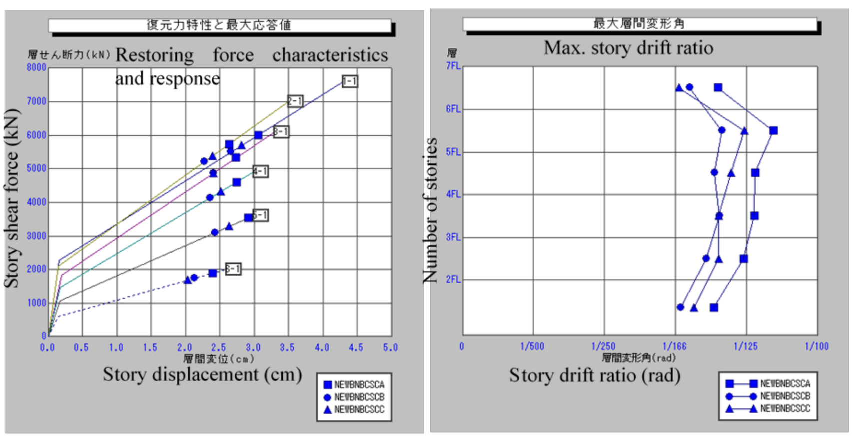


X direction Y direction

Fig. 13 - Relation between response base shear force and story deflection Source: JICA project

In addition, a building site was supposed as located in Zone 4 (Z= 0.36) of BNBC, where design PGA for dynamic analysis is 0.276G (= 2/3x 0.36G x 1.15), which is 1.8 times of that of Zone 2, and the result of X direction is shown in Fig.14. The result showed that the response was within the yield strength. The result of Y direction was also within the yield strength (response ductility ratio < 0.9).

Furthermore, typical existing waves (El-centro NS, Taft EW and Hachinohe NS) with PGA 0.276G were inputted for the comparison purpose. The result is shown in Fig. 15 for X direction, and similar results were obtained.



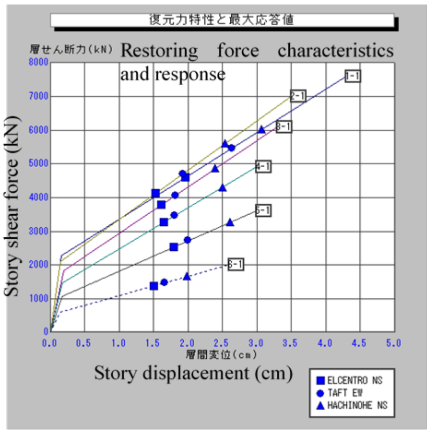


Fig. 14 - Relation between response base shear force and story deflection for X direction (left, center)

Fig. 15 - Responses by typical 3 waves with input of PGA 0.276G for X direction Source: JICA project

5. Retrofit construction

A uniting work to the existing columns on both sides of the new six-story RC shear wall and reinforcing bar/ concrete work, RC jacketing work on existing columns, reinforcement work of foundation footings, etc. were provided on existing lower three stories as shown in Fig. 16. Non-shrink grout mortar was provided at the top of each RC shear wall with additional re-bars to prevent the split failure of the concrete. Upper three stories were constructed based on the BNBC like a new building.

a) New RC shear wall and strengthening of shallow foundation, b) Column jacketing, c) Structural work

Fig. 16 - Retrofit construction Source: JICA project

6. Conclusion

The existing three-story RC building has been effectively extended vertically and seismically upgraded by the adoption of a dual structural system using six-story RC shear walls and RC jacketing on columns. In order to assess the seismic performance, a pushover analysis and a time-history response analysis were conducted focusing on the collapse mode of the structure, and the ductility of multi-story RC shear wall and RC moment frame. As a result, the effectiveness of this system has been shown, and the existing beams were assessed not required seismic retrofit in this case. The story deflection angle (story drift ratio) was suppressed to less than 1/200 rad. against the dynamic seismic design load of the BNBC. The damages to the non-structural walls, which are brittle brick walls, would be minimized. Further issues on the discussion would be the retrofit design on the desirable combination of multi-story RC shear wall and RC moment frame with respect to the strength and ductility, and the proper judgement on existing beams when the column jacketing is done.

7. Acknowledgements

This study was conducted as a part of the international technical cooperation projects (CNCRP and BSPP) between Public Works Department (PWD), Ministry of Housing and Public Works of Bangladesh and Japan International Cooperation Agency (JICA).

8. References

[1] Housing and Building Research Institute (HBRI): Part 6 Structural Design, Bangladesh National Building Code (BNBC) 2015 Draft, Bangladesh.

[2] Public Works Department (PWD), JICA CNCRP Project (2011~2015): Seismic Evaluation Manual and Retrofit Design Manual for Existing RC Buildings, Bangladesh.

[3] The Japan Building Disaster Prevention Association: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings 2001, and Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings 2001, English version, 1st, Japan.

[4] A. Inoue et al.: Suggested modification of Japanese Seismic Evaluation and Retrofit Design for RC buildings for its application in Bangladesh, *16WCEE* Chile.

[5] Rafiqul Islam et al.: Retrofitting design and construction of mixed type structure with RC frame and masonry in Dhaka, Bangladesh, *16WCEE* Chile.

[6] Anup K. Halder et al.: Seismic retrofitting of a garment factory building in Bangladesh, *New Technology for Urban Safety of Mega Cities in Asia, Nepal Kathmandu 2016*, Nepal.

