SEISMIC STRENGTHENING OF A SCHOOL BUILDING DAMAGED BY THE 2004 NIIGATA-CHUETSU EARTHQUAKE

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ABSTRACT

This paper deals with the strengthening of a school building damaged by the Niigata-Chuetsu Earthquake (Magnitude 6.8) of 2004. The building locates in the seismic zone where the intensity is registered 6+ on the 7-grade Japanese intensity scale.
The building was a 3-story reinforced concrete frame with total floor area of 2369m^2 constructed in 1968. Numerous shear cracks were observed in columns and bending cracks in beams due to the earthquake and some columns were broken and failed completely.
"Steel plate embedded reinforced concrete brace" was applied as a strengthening method and connected to the beam-column-beam joints together in order to increase the strength of the building. "Steel plate embedded reinforced concrete brace" was connected by post-installed anchors to the beam-column joints with the embedded steal plate. The joints were covered with concrete as well.

KEYWORS: seismic strengthing, earthquake, damagad building

1. INTRODUCTION

The Niigata-Chuetsu Earthquake occurred on the 23rd October 2004. It caused the damages to reinforced concrete structures such as school building. This paper deals with the extent of damage, seismic performance of the damaged building and subsequent strengthening.
The building was a 3-story school with a penthouse located 20 km from the epicenter of the earthquake. The damage was minor while the inspection for the seismic performance of the building required repairs and strengthening for the structural members.
The seismic performance was evaluated on the basis of the Seismic Inspection Standard for Reinforced Concrete Buildings 2001 [1] and the necessary seismic strengthening design were performed.

2. OUTLINE OF THE BUILDING AND DAMAGES

2.1 The building

The building, constructed in 1968, was a 3-story reinforced concrete school building with a pile foundation. The building was 11.3 m high with a floor area of 2369.0 m^2 and built on a partial embankment as it was at the top of a hill. Major elevation, plan and section of the framing components are shown in Fig. 2.1. A view of Y0 direction is shown in Photo 2.1 and the embankment was made at X9 side. Longitudinal direction of the building was a pure

Photo.2.1 View of Existing Building

moment resisting frame structure while span direction was a rigid frame combined shear walls. Columns of Y2 direction are a short column with an inner height h_0 of 2400mm and a depth D of 1200mm (h_0/D=2.0) as shown in Fig. 2.1. Multi-story shear walls were the main components of span direction while the wall distribution was not uniform so as to result in a large eccentricity at the first floor. Also arrangement of walls and openings were different from the original design sacrificing the designed seismic performance. Design strength of concrete, Fc was 17.6 N/mm^2 but measured values ranged from 13.1 to 17.0 N/mm^2 at the first floor, 11.9 to 19.8 N/mm^2 at the third floor and 15.1 to 15.2 N/mm^2 at the second floor, most are significantly lower than the design strength. Grade of the steel bar was round steel (σ_y=294N/mm^2).

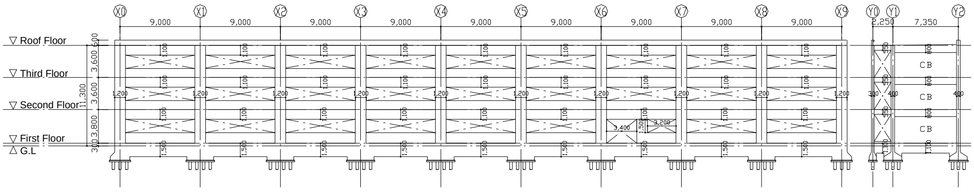

Fig.2.1 Elevation and Section of Typical Side

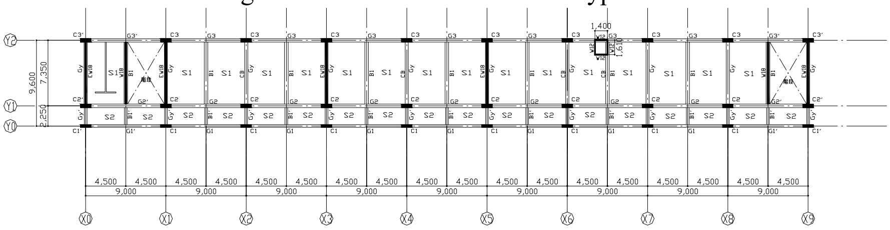

Fig.2.2 Plan of Typical Floor

2.2 Damages

Degree of seismic damage to buildings has been estimated with reference to the Damage Evaluation Criteria and the Technical Recommendation for Recovery [2]. The main damage as follows.

2.2.1 Damage to the upper structure

A number of short columns at Y2 axis exhibited shear cracks while bending cracks were found in beams. The shear walls in the span direction also showed shear cracks. The largest damage was found in columns of the first floor. Of thirty columns, two columns marked degree IV, crack width larger than 2 mm, as shown in Photo. 2.2, and one column marked degree III, crack width between 1 to 2 mm. Hence the degree of damage to the upper structure was worked out to be “Slight”.

Photo.2.2 Damage to Column

2.2.2 Damage to the foundation

After the earthquake, an uneven settlement was found centering around X9 axis leading to excavation of piles and level surveys of the first and second floors. The pile inspection showed a bending crack of 0.2 to 2.0 mm wide at 500 to 600 mm from the top of a pile as shown in Photo. 2.3.

The level survey showed a settlement from Y0 to Y2 axis. A large drift angle q was found at X5 structural plane between Y0 and Y1 at the first and the second floors, θ=1/225 and θ=1/188 respectively. Settlement along the longitudinal direction was also recognized at Y0 structural plane X4 direction but the drift angle was as low as1/1800 to 1/1500. The largest drift angle was θ=1/900 found in the first floor at Y0 structural plane between X7 and X8 direction. The maximum settlement of the first floor was 31 mm. Hence the degree of damage to the foundation was worked out to be “Moderate”.

Photo.2.3 Bending Crack of Pile (2mm Width)

3. SEISMIC PERFORMANCE BEFORE EARTHQUAKE

3.1 Evaluation method and assumptions

Seismic performance of the building for each floor and direction was evaluated in terms of seismic index of structural elements *Is*. [1] which is defined as

$Is = E_0 \cdot S_D \cdot T$

where E_0: seismic index of basic structural performance including strength index (*C*) and ductility index (*F*), S_D: irregularity index representing shape complexity and rigidity distribution and *T*: aging index evaluating effects of crack and deformation on seismic resistance.

Assumptions to calculate *Is* are as follows:

(1) Analytical aspect

Strength indexes (*C*) and ductility index (*F*) in E_0 index are based on vertical members, so-called secondary diagnosis method.

(2) Compressive strength of concrete was based on the core strength (from 12.1 to 18.8 N/mm^2) and adopted 13.5 N/mm^2, 12.4 N/mm^2 and 13.9 N/mm^2 for the first floor, second floor and third floor respectively. The yield strength (σ_y) of steel reinforcement was 294 N/mm^2.

(3) Effects of seismic damage

Reduction of seismic performance of damaged structural members were not taken into account but was reflected in the aging index of the entire building.

(4) Estimation of column strength

Bending strength of column was estimated at the face position.

(5) Strength of multi-story shear wall

External forces for the multi-story shear wall follow Ai distribution taking into account rotation of the foundation, so-called the third diagnosis.

3.2 Results of seismic performance evaluation

Distributions of the retained basic seismic index E_0 and seismic index of structural elements *Is* are shown in Fig. 3.1～3.2. Seismic performance of the building, either longitudinal (X) or span (Y) direction, is thus evaluated as follows.

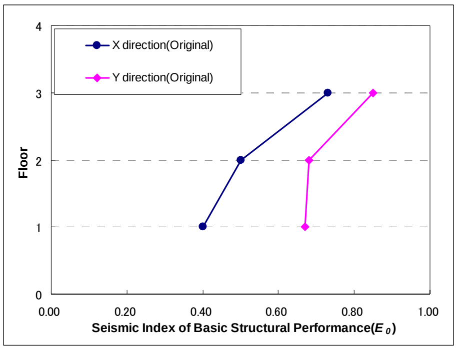

Fig.3.1 E_0－Index

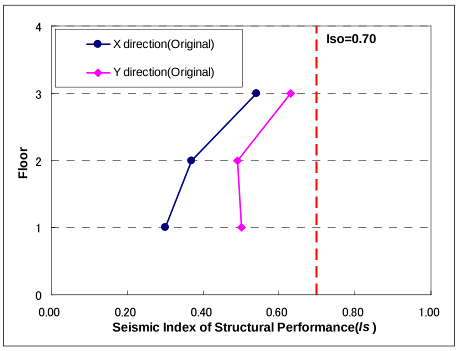

Fig.3.2 Is－Index

(1) Longitudinal direction (X)

A major fracture mode of each floor was shear fracture of the short columns whose brittleness leads to an ultimate ductility index (*Fu*) of 0.8. Hence the index E_0 ranges from 0.40 to 0.73. Taking into account for the irregularity index of S_D=0.93 and the aging index of *T*=0.81, the seismic index *Is* of the building is worked out to be from 0.30 to 0.54, which is lower than seismic evaluation index of *Iso*=0.7. The building is thus of poor seismic performance at entire floor.

(2) Span direction (Y)

Fracture mode of the multi-story shear wall without openings is rotation of foundation (*F*=3.0) while that with openings is shear facture (*F*=1.0). Thus the ultimate ductility index (*Fu'*) in estimating E_0 yields 1.0 and E_0 ranges from 0.67 to 0.85. Finally, the seismic index *Is* becomes 0.49 to 0.63 which is less than *Iso*=0.7. The building is thus of poor seismic performance at entire floor.

4. SEISMIC STRENGTHENING DESIGN

4.1 Targeted performance

Targeted value of seismic strengthening ($_{R}Iso$) was set to be 0.7 which is the same as seismic evaluation index (Iso).

4.2 Strengthening course

Columns and beams damaged by the earthquake must be repaired and therefore the following seismic strengthening plan was formulated.

(1) Longitudinal (X) direction

Structural slits are introduced to the spandrel walls at Y0 axis and reinforced concrete braces with embedded steel plate, hereafter referred to as S-RC braces, capable of reinforcing columns and beams at the same time are added to Y2 axis after removing the extremely brittle columns that may act as a main structural element.

(2) Span (Y) direction

Reinforced concrete with additional shear walls, hereafter referred to as infilled RC wall, are added to increase strength and improve the eccentricity after removing concrete block walls and damaged walls with opening.

4.3 Reinforcement design

Outline of the strengthening plan is shown in Fig. 4.1～4.2. Major strengthening methods are as follows.

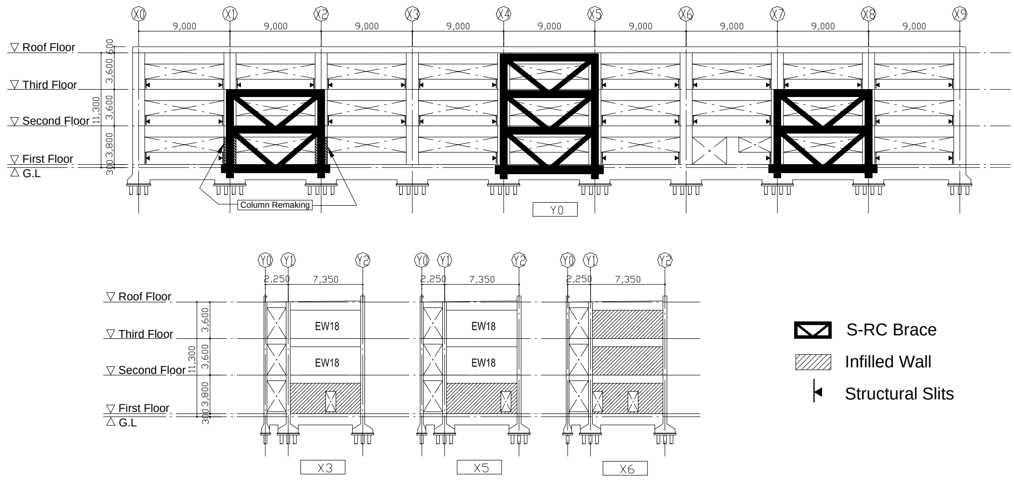

Fig.4.1 Arrangement of Retrofitting Members in Elevation and Sections

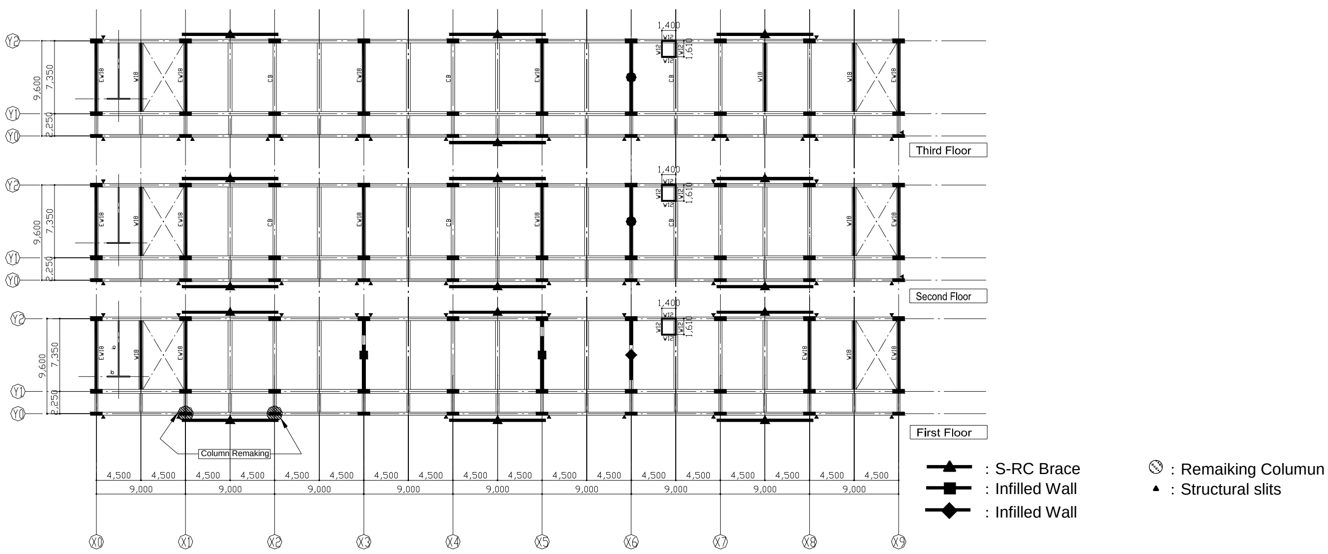

Fig.4.2 Arrangement of Retrofitting Members in Plans of Each Floor

(1) S-RC brace (PITA-column)

S-RC braces were newly introduced in the X axis: six braces for first and second floor and four for third floor. This method is a retrofitting of reinforced concrete brace with embedded steel plate to the outside of the structural plane with drilled anchors. The thickness of the plate was 22 mm and anchor bolt attached to the plate were D19 bonded anchor with an effective embedding depth of 12da (da: anchor diameter). Reinforcing columns and beams at the same time, the method was considered as an effective measure to strengthen the building with reduced concrete strength. Strength of the S-RC brace was determined on the basis of foundation rotation mode because it was designed as multi-story and single span, while its ductility index (F) is evaluated as 1.0(shown Fig.4.3～4.5).

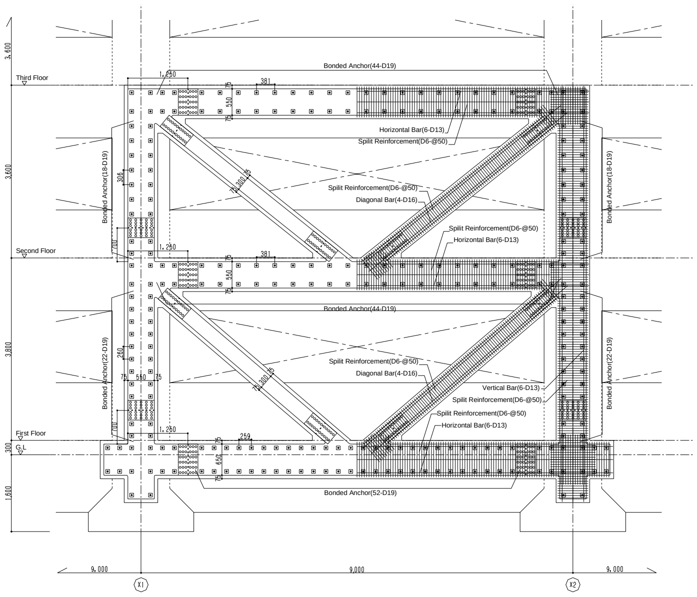

Fig.4.3 Details of R/C Brace with a Built-In Steel Plate (S-RC Brace)

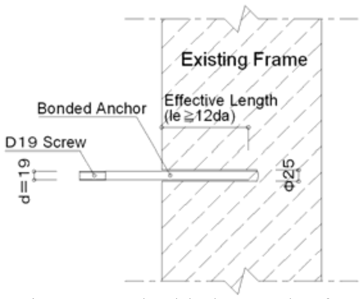

Fig.4.4 Embedded Length of Bonded Anchor Used in S-RC Brace

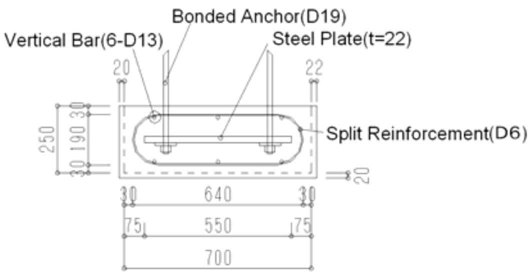

Fig.4.5 Existing-Additional Column Connection

(2) Infilled RC wall

After the damaged RC walls and concrete block walls were removed, additional RC shear walls were constructed at Y direction, 3 walls at the first floor and 1 at second and third floor, the multi-story configuration was remained. The additional RC shear walls at the first floor have an opening with a thickness of 300 mm and were reinforced with D13 steel bars at a horizontal and vertical spacing of 200 mm, while there are no opening but only 200 mm thick walls at second and third floors. They were reinforced with D10 steel bars at a horizontal and vertical spacing of 200 mm. The drilled anchor bolts used to connect existing columns and beams of the first floor were D22 bonded anchor with an effective anchor depth of 7da while those used in second and third floor were D19 bonded anchor type with an effective anchor depth of 7da.

Because the infilled RC shear walls were designed as a multi-story single span, strength was estimated assuming foundation rotation mode, thus the ductility index (F) can be estimated as 3.0(shown Fig4.6～4.7).

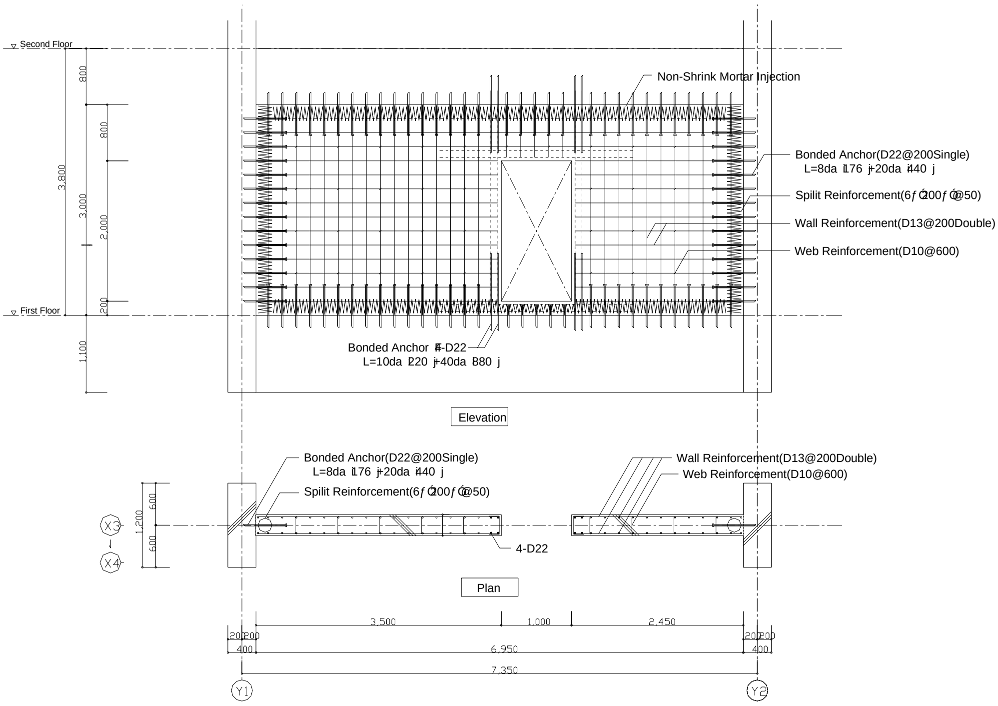

Fig.4.6 Details of Retrofitting RC Shear Wall with Opening

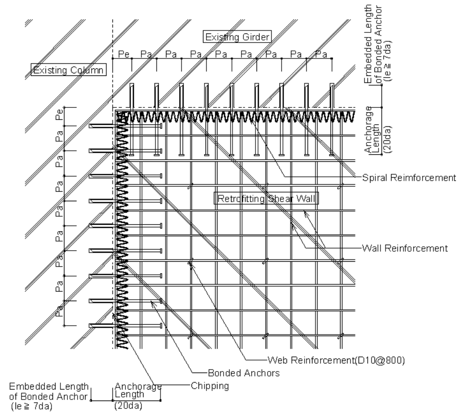

Fig.4.7 Detail of Retrofitting Shear Wall

(3) Structural slits
Mainly for X direction, structural slits were introduced, 19 at first floor, 15 at second floor and 18 at third floor. These slits were a partial groove with a height of the spandrel wall, width of 30 mm and depth of 70 mm.

5. SEISMIC PERFORMANCE AFTER STRENGTHENING

Distributions of indexes E_0 and *Is* before and after the strengthening are shown in Fig. 5.1～5.2 where it is demonstrated that the strengthening described in section 4 was able to increase index E_0 1.2 to 2.3 times greater than before for X direction and 1.2 to 1.4 times greater than before for Y direction. Index *Is* was also increased 1.4 to 2.7 times greater than before for X direction and 1.5 to 1.7 times greater than before for Y direction. Thus the Index *Is* showed remarkable increase exceeding *Iso*=0.7 and the building now has a satisfactory seismic performance

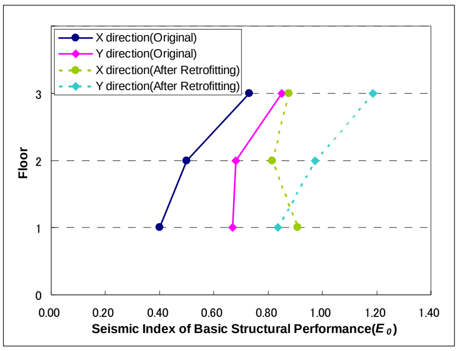

Fig.5.1 Comparison Between Before and After Retorofitting E_0-Index

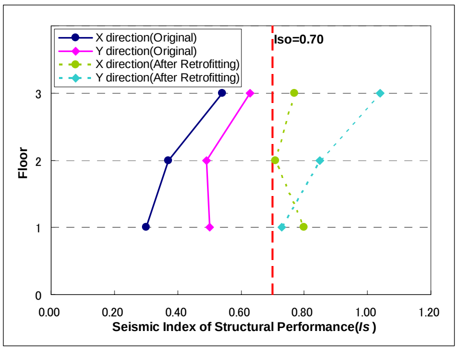

Fig.5.2 Comparison Between Before and After Retorofitting Is-Index

6. CONCLUSIONS

Seismic evaluation was performed for a building damaged by the 2004 Niigata-ken Chuetsu Earthquake. The damage was slight in upper structure and moderate in the foundation resulting in reduction of seismic performance with a seismic index of structural elements *Is* lower than assessed index of *Iso*=0.7 for X and Y directions of all the floors.
Seismic strengthening was carried out, and S-RC brace and structural slits were introduced in X direction and reinforced concrete shear walls were added in Y direction improving ductility of the structure.
Seismic evaluation for the strengthened building confirmed the *Is* index for X and Y directions of all the floors exceeded the assessed value of *Iso*=0.7(shown Photo.6.1).
During the Niigata-Chuetsuoki Earthquake (Magnitude 6.8) of 2007, with the intensity of 6+, the strengthened building was not subjected to any damage.

Photo.6.1 View of Building after Retrofitting

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