# EVALUATION & STRENGTHENING OF A EXISTING REINFORCED CONCRETE SCHOOL BUILDING

M. Takaki (I)
A. Ikeda (II)
Presenting Author: M. Takaki

## SUMMARY

This paper presents the results of evaluation of the seismic safety and strengthening of a existing 3 storied reinforced concrete school building. At first the seismic safety of the exising building was evaluated and judged unsafe. So it was strengthened. After the strengthening the seismic safety of the building was evaluated again by calculating ultimate strength and nonlinear response analysis. Also material tests, geological inspection and measurements of microtremor were done. Microtremor was measured before and after the strengthening. As the result it was confirmed that the strengthening was effective enough that the building has obtained enough resistibility against severe earthquakes.

## INTRODUCTION

In Japan, it was recently pointed out that there might occur a strong earthquake of Magnitude 8 in Tokai district in the future of no more than 20 years. Then in that area we are in urgent need of evaluation of the seismic safety and strengthening of existing buildings such as schools, public services and the like. As a part of the above works, evaluations of the seismic safety and strengthenings were done in several existing buildings. This paper introduces the case of 3 storied reinforced concrete school building.

## OUTLINE OF THE BUILDING AND ITS SURROUNDING GEOLOGY

The building in concern which was constructed in 1965 is a 3 storied reinforced concrete school building with straight plan of 99.8m x 9.7m. The 1st story plan of the building is shown in Fig. 1. The building is composed of two parts which are called A1 Block and A2 Block. They are connected with expansion joint. A1 Block has 5 spans in the longitudinal direction (the direction X) and 2 spans in the transverse direction (the direction Y). A2 Block has 9 spans in the direction X and 3 spans in the direction Y. The structure of the building is open frame in the direction X and frame-wall in the direction Y. The foundations are individual foundations.

Soil profile is shown in Fig. 2. The subsoil consists of sand and gravel. N values of the sand layers at 4m and above were 5-50 and those of

(I) Research Engineer, Earthquake Engineering Laboratory Technical Research Institute, TAISEI CORPORATION, JAPAN.

(II) Manager of Structural Department, Engineering Doctor, Technical Research Institute, TAISEI CORPORATION, JAPAN

gravel or dense sand layers below 4m were more than 50.

## EVALUATION OF THE EXISTING BUILDING

Evaluation was done according to the guide line specified by Ministry of Construction. Is (Structural Index) values were calculated by the method and were compared with the specified criterion of 1.0. Here, Is value corresponds to the ultimate shear coefficient of the 1st floor in case of a rigid and brittle building.

Is values of the existing building were shown in Table 1. As for A1 Block Is values in the direction Y were as high as 1.27 - 1.30. But Is values in the direction X were 0.47 - 0.75 and they could not pass the specified criterion of 1.0. As for A2 Block Is values in the direction Y were 0.67 - 1.15 and those in the direction X were 0.49 - 1.18. Is values of the 1st and 2nd story in the both horizontal directions were smaller than 1.0, and the values of the 1st story were the smallest in each direction.

As the result of evaluation, it turned out neccessary to strengthen all three stories of A1 Block in the direction X and 1st and 2nd stories of A2 Block in the both horizontal directions.

## STRENGTHENING

In consideration of the results of evaluation, strengthening was done by constructing shear walls of 20 cm thickness in the existing frames. In the direction X spandrel walls on the north side which made the columns short and brittle were removed and 27 panels of shear walls with small opening were newly constructed instead. In the direction Y thin walls (non-structural walls) were removed and instead of them 9 panels of shear walls were newly constructed.

The method of constructing new shear walls is as follows.

(1) Remove spandrel walls from the existing frames.

(2) Set mechanical anchors on the surface of the existing frames to connect newly constructed walls and existing frames (Fig. 3).

(3) Place reinforcing bars. The spiral bars are necessary to prevent wall concrete split (Fig. 4).

(4) Assemble forms with injection and overflow nozzles for concrete.

(5) Pump in concrete with adequate pressure.

The cover concrete to guard the waterproof layer on the roof was removed and waterproof was exchanged to an exposed type in order to minimize the weight increase. Consequently 189t was reduced, which compensated a part of weight increase associated with the strengthening. Final increase of weight was 742t.

## TESTS AND MEASUREMENTS

Material tests were done on the specimens of the concrete and steel bars sampled from the existing building. Compression tests and oxidation tests were done on the existing concrete and tension tests were done on steel bars. Microtremor was also measured on the building before and after the strengthening.

Compression tests of the existing concrete.

Total 18 pieces of specimens of the concrete were cut out from spandrel walls in each stories of both Blocks with concrete-core-drill. The diameters and the heights of the specimens were all 10 cm and 20 cm respectively.

As the result of compression tests, the maximum compressive strength was 593 $kg/cm^2$, the minimum was 224 $kg/cm^2$ and the average of them was 355 $kg/cm^2$. These values exceeded the supposed design compressive strength. Young's maduli were in the range from $2.17 \times 10^5$ $kg/cm^2$ to $3.32 \times 10^5$ $kg/cm^2$ and larger than the supposed design value.

Degree of oxidation

18 pieces of specimens were cut out from spandrel walls. They were spraied with phenolphtalein solution and the uncolored depth from the surface were measured.

Oxidized depths were in the range from 0 to 3 mm, and no problem was found.

Tension tests of steel bars

Tension tests were done on the 18 specimens of the steel bars cut out from the removed spandrel walls. The diameters of the bars were all 9 mm.

The maximum yield strength was 40 $kg/mm^2$, the minimum was 33 $kg/mm^2$ and the average of them was 36.2 $kg/mm^2$. The maximum of ultimate strength was 53 $kg/mm^2$, the minimum was 39 $kg/mm^2$ and the average of them was 47.3 $kg/mm^2$.

Measurements of microtremor

Microtremor was measued on each block of the building before and after the strengthening. Moving-coil type displacement transducers were set on each story and ground as shown in Fig. 1.

The maximum displacements, torsion ratios and rocking ratios at roof floor before and after the strengthening are shown in Table 2. Torsion ratio in A1 Block after the strengthening was about five times as much as before and the ratios in A2 Block after the strengthening was about two times as much as before. But the maximum ratio was about 15% which seemed not so significant. Rocking ratios became about 1.5 times as much as before owing to strengthening.

The Fourier amplitude ratios of the roof floor to the 1st floor in A2 Block before and after the strengthening are shown in Fig. 5. The natural periods obtained by these ratios are shown in Table 3. The natural period became about 15% shorter after the strengthening which indicates that the strengthening increased the stiffness of the building.

## EVALUATION AND RESPONSE ANALYSIS OF THE STRENGTHENED BUILDING

Evaluation was again done on the strengthened building according to the same method before the strengthening. The results are shown in Table 1. The extremely brittle columns were removed by the strengthening. The Is values in both directions of both strengthened blocks were in the range from 1.02 to 1.55 and exceeded the specified criterion of 1.0. It was judged that the strengthened building had enough seismic resistibility.

Nonlinear response analisis was also done using the restoring force-displacement characteristics of each stories composed of those of members. 3-degree-of-freedom lumped mass models were used to represent the building with three kinds of equivalent shear springs. In addition to the fixed-base condition swaying freedom was also considered as shown in Fig. 6.
The equivalent shear springs were three types: bending columns, shear walls and bending walls. The nonlinear restoring force-displacement characteristics of each members were built in consideration of shear and bending cracks and yeild.

Natural period and participation function of A1 Block in the direction X and Y are shown in Fig. 7. A2 Block had the same frequency properties as A1 Block. The natural periods of A2 Block are shown in Table 4. The values of participation function at the top of the buildings in each case were about 1.3 - 1.6. The lumped mass models with these natural properties were subjected to earthquake ground motions: EL CENTRO 1940 NS, TAFT 1952 EW, HACHINOHE 1968 EW, whose maximum accelerations were normalized to 450 Gal. Damping values of the building were assumed as 5% at the fundamental period and frequency proportional in higher modes. Damping factors of soil were assumed as 27 - 42% based on the half-space theory.

Results of the nonlinear response are shown in Table 5. The acceleration response magnifications at the top were 1.1 - 3.5. The maxima of story deflection angles were 1/2800 - 1/250. Comparing the results of fixed base model and sway model, there was no difference about accelerations. The story deflection angles of sway model were 1/2.7 - 1/1.1 of those of fixed base model. This is due to sway displacement which occupies 25 - 60% of the total displacement. As a typical example, the result of A1 Block sway model in the direction X are shown in Fig. 8. It can be judged that the building would not be severely damaged though shear cracks and bending cracks might develop in the members if the strengthened building are exposed to very severe earthquake motion of 450 Gal in the future.

## CONCLUSIONS

From the results of the studies on the natural properties and seismic resistibility of the building before and after the strengthening, the following conclusions were drawn.

(1) It was comfirmed that the stiffness and seismic resistibility of the building were improved by the strengthening.

(2) According to the nonlinear response analysis to strong earthquake ground motion, it was judged that the seismic safety was secured thongh sheare cracks and bending cracks might develop in the members in case of very severe earthquakes.

Table 1 Is Values of Before and After the Strengthening

| Block | Direction | Story | Before | After | Block | Direction | Story | Before | After |
|---|---|---|---|---|---|---|---|---|---|
| A1 | X | 3 | 0.75 | 1.29 | A2 | X | 3 | 1.18 | 1.42 |
| | | 2 | 0.55 | 1.03 | | | 2 | 0.68 | 1.15 |
| | | 1 | 0.49 | 1.06 | | | 1 | 0.49 | 1.02 |
| | Y | 3 | 1.30 | 1.40 | | Y | 3 | 1.15 | 1.55 |
| | | 2 | 1.27 | 1.29 | | | 2 | 0.77 | 1.14 |
| | | 1 | 1.28 | 1.30 | | | 1 | 0.67 | 1.09 |

Table 2 The Results of Measurement of Microtremor

| Direction | Strength-ening | | A1 Block | | A2 Block | |
|---|---|---|---|---|---|---|
| | | | IF | RF | IF | RF |
| X | Before | Max. Disp. ($\mu$m) | 0.603 | 0.692 | 0.332 | 0.348 |
| | | Torsion Ratio | 0.029 | 0.025 | 0.086 | 0.045 |
| | After | Max. Disp. ($\mu$m) | 0.222 | 0.271 | 0.357 | 0.404 |
| | | Torsion Ratio | 0.149 | 0.127 | 0.082 | 0.085 |
| Y | Before | Max. Disp. ($\mu$m) | 0.234 | 0.300 | 0.213 | 0.293 |
| | | Rocking Ratio | 0.460 | | 0.415 | |
| | After | Max. Disp. ($\mu$m) | 0.249 | 0.311 | 0.328 | 0.581 |
| | | Rocking Ratio | 0.658 | | 0.602 | |

Table 3 Natural Period Before and After the Strengthening
(By Microtremor)

| Direction | A1 Block | | | A2 Block | | |
|---|---|---|---|---|---|---|
| | Before | After | After/Before | Before | After | After/Before |
| X | 0.14 sec.<br>0.115 | 0.13 sec.<br>0.115 | 1/1.08 ~ 1.0 | 0.14 sec.<br>0.15 | 0.12 sec.<br>0.13 | 1/1.17 ~ 1/1.15 |
| Y | 0.16 | 0.145 | 1/1.10 | 0.165<br>0.13 | 0.14<br>0.11 | 1/1.18 |

Table 4 Natural Period of A2 Block
(By Calculation)

| Model | Direction | 1st (sec) | 2nd (sec) | 3rd (sec) |
|---|---|---|---|---|
| Fixed -base | X | 0.127 | 0.0540 | 0.0336 |
| | Y | 0.201 | 0.0852 | 0.0558 |
| Sway | X | 0.220 | 0.0911 | 0.0514 |
| | Y | 0.250 | 0.126 | 0.0790 |

Table 5 Maximum Response Values

<table>
<tr><th>Model</th><th>Block</th><th>Direction</th><th>Item</th><th>EL CENTRO</th><th>TAFT</th><th>HACHINOHE</th></tr>
<tr><td rowspan="8">Fixed -base</td><td rowspan="4">A1</td><td rowspan="2">X</td><td>Max. Acc. (gal)</td><td>843</td><td>967</td><td>704</td></tr>
<tr><td>Max. Angle</td><td>1/1146</td><td>1/408</td><td>1/1472</td></tr>
<tr><td rowspan="2">Y</td><td>Max. Acc. (gal)</td><td>566</td><td>591</td><td>521</td></tr>
<tr><td>Max. Angle</td><td>1/700</td><td>1/308</td><td>1/458</td></tr>
<tr><td rowspan="4">A2</td><td rowspan="2">X</td><td>Max. Acc. (gal)</td><td>555</td><td>651</td><td>499</td></tr>
<tr><td>Max. Angle</td><td>1/408</td><td>1/286</td><td>1/413</td></tr>
<tr><td rowspan="2">Y</td><td>Max. Acc. (gal)</td><td>680</td><td>745</td><td>632</td></tr>
<tr><td>Max. Angle</td><td>1/639</td><td>1/246</td><td>1/486</td></tr>
<tr><td rowspan="8">Sway</td><td rowspan="4">A1</td><td rowspan="2">X</td><td>Max. Acc. (gal)</td><td>725</td><td>801</td><td>621</td></tr>
<tr><td>Max. Angle</td><td>1/2788</td><td>1/1096</td><td>1/1925</td></tr>
<tr><td rowspan="2">Y</td><td>Max. Acc. (gal)</td><td>1537</td><td>930</td><td>1577</td></tr>
<tr><td>Max. Angle</td><td>1/811</td><td>1/339</td><td>1/555</td></tr>
<tr><td rowspan="4">A2</td><td rowspan="2">X</td><td>Max. Acc. (gal)</td><td>506</td><td>589</td><td>559</td></tr>
<tr><td>Max. Angle</td><td>1/664</td><td>1/481</td><td>1/623</td></tr>
<tr><td rowspan="2">Y</td><td>Max. Acc. (gal)</td><td>739</td><td>777</td><td>709</td></tr>
<tr><td>Max. Angle</td><td>1/823</td><td>1/348</td><td>1/508</td></tr>
</table>

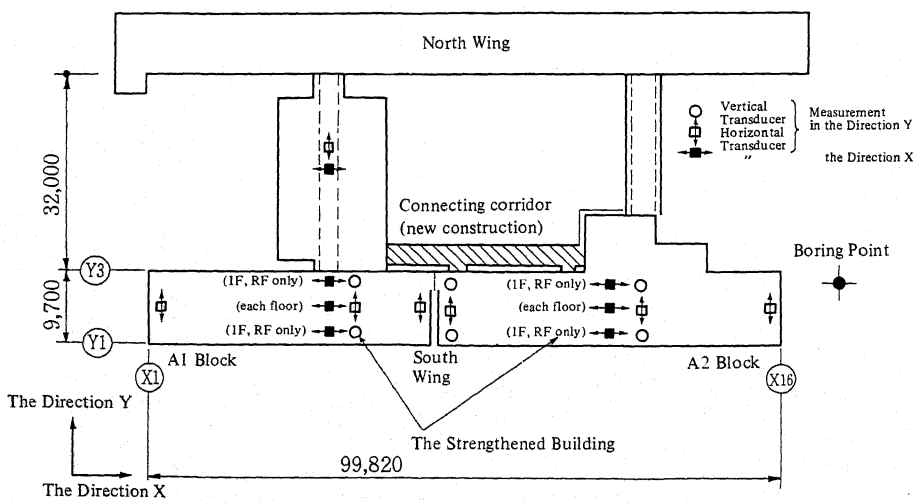



Fig. 1 The Plan of the Strengthened Building & Measurement Points of Microtremor

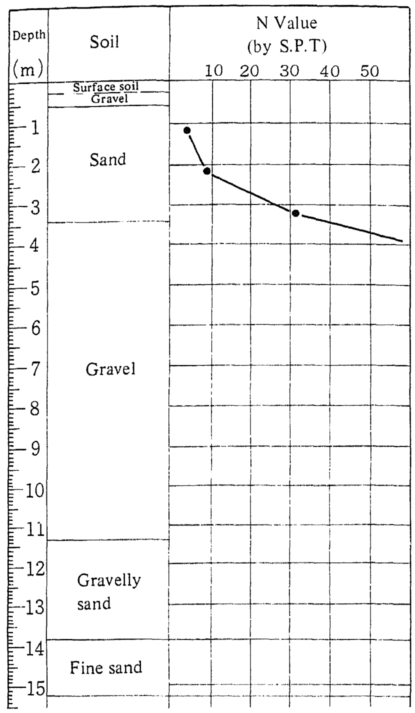



Fig. 2 Soil Profile of Boring Point

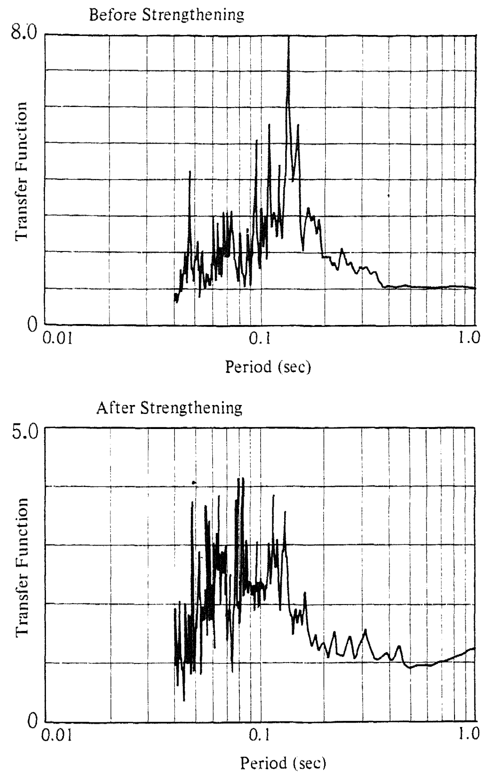



Fig. 5 Spectral Rario

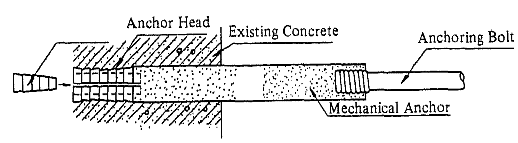



Fig. 3 Mechanical Anchor

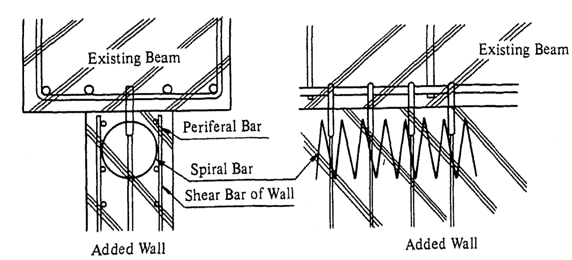
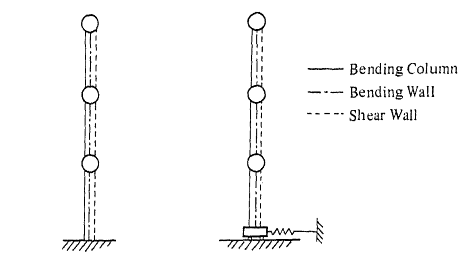
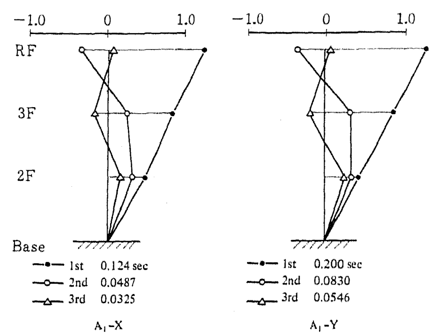



Fig. 4 Joint of the Existing Beam & Added Wall

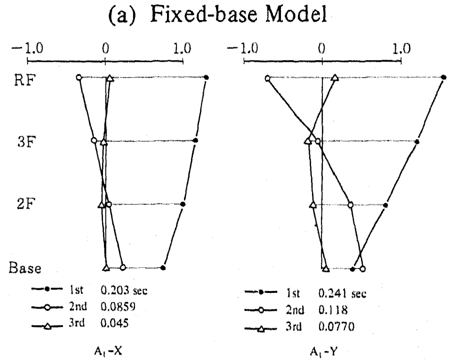



(a) Fixed-base Model (b) Sway Model

Fig. 6 Lumped Mass Models




(a) Fixed-base Model




(b) Sway Model

Fig. 7 Natural Periods and Participation Functions

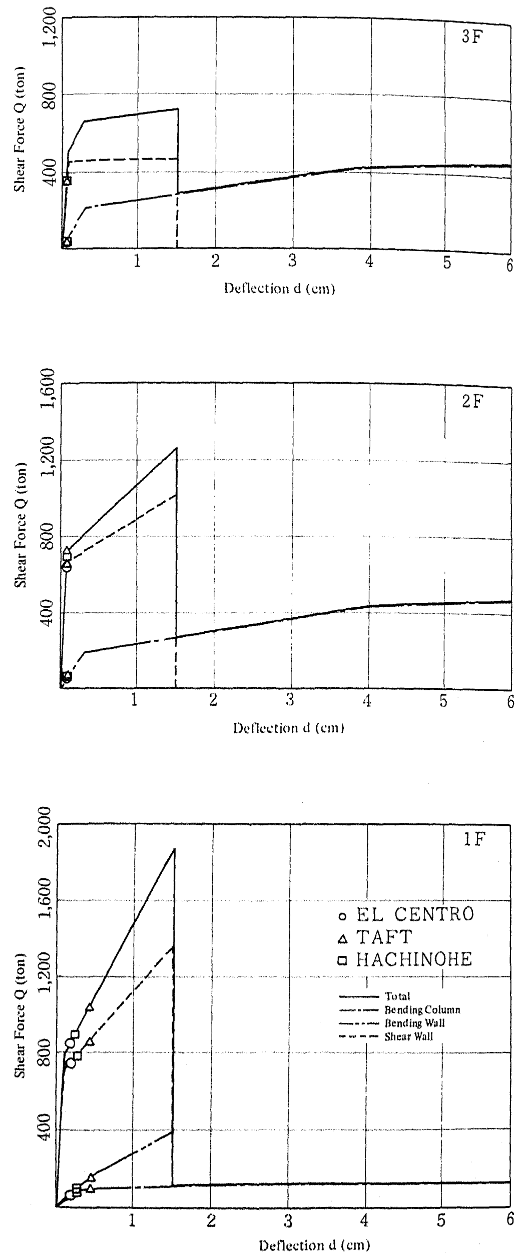



Fig. 8 Nonlinear Response of the Strengthened Structure