ENGINEERING NONLINEAR STRUCTURAL RESPONSE IN RC PIERS: A COMPARISON BETWEEN FULL SCALE EXPERIMENTS AND FIBER ELEMENT MODEL ANALYSIS

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Introduction

The 1995 Hyogoken Nanbu Earthquake caused a great deal of damage to bridge structures. As a result, the Japan Road Association' s Specifications for Highway Bridges was modified the following year. Modifications included the introduction of the stress-strain relationship in concrete and changes to the calculation method for the horizontal force-displacement relationship in RC piers. At the same time, the stress-strain relationship of concrete was introduced in consideration of the confinement effects of tie reinforcements. Moreover, in order to improve ductile properties, the details oreinforcement arrangement were coded and the shear resistance was revaluated to take dimensional effects into consideration.

Even though several experiments were carried out on small-scaled models, the large-scaled model's experimental data was necessary to make safety judgments on the bridge design

E-Defense, the world's largest shake table experiment facility, is owned by the NIED, a non-profit organization, and was constructed to help clarify damage mechanisms of structures affected by the Hyogoken Nanbu Earthquake. Since many of the damage phenomena have not been understood, several experiments were performed.

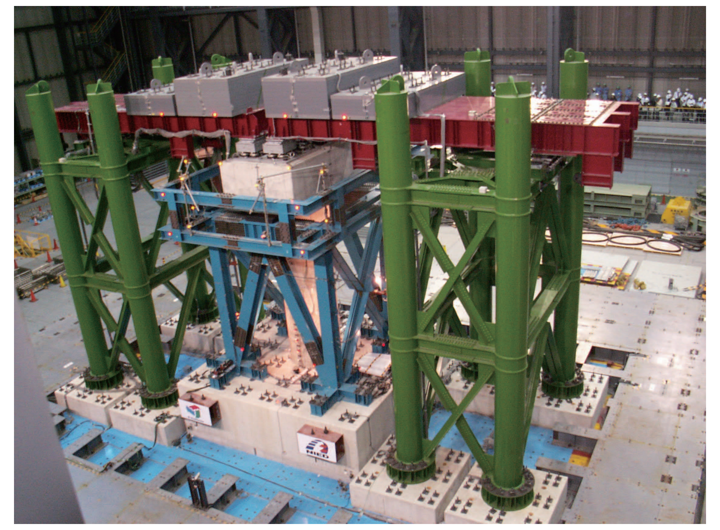
This paper reviews the completed C1-2 experiments and introduces a results comparison of the analysis model and the experiment in the anticipation of blind analysis contest. The modeling tool and model validity is also verified

Experimental Summary

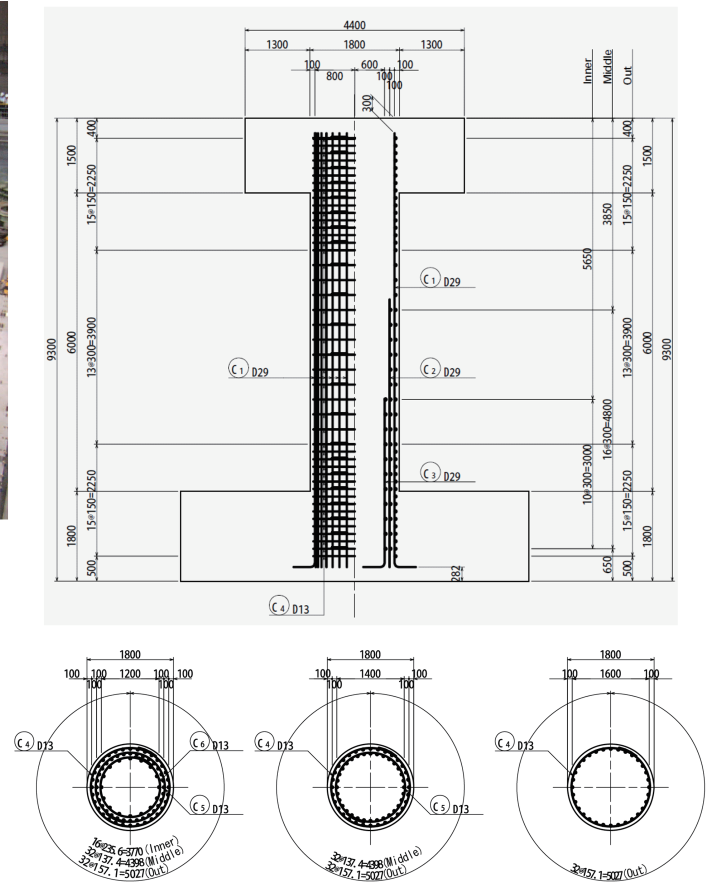
Shape and Materials

Figure 1 shows the C1-2 experiment set and Figure 2 shows the dimension and reinforcement arrangement of the pier.

The C1-2 height from the pier base to the cap beam upper position is 7.5m and its diameter is 1.8m. All main reinforcements are D32, and their arrangments are 2.5 layers from the pier base to 1.87m, 2 layers from 1.87m to 3.87m, and 1 layer from 3.87m to the pier top. The tie reinforcements are D13 with 300mm intervals for the internal, middle, and outside layers. But the outer layer tie at the part from the base to 0.95m and from 4.85 to the top is condensed with interval



▲Figure1



▲Figure 2

150mm. The reinforcement material is SD345 and its tension strength based test is 372MPa. Test points were also set on the concrete surface at a position 3.0m from the pier base. The concrete strength on the test day according to tabular tests was 33.1MPa for the lower part and 28.4MPa for the upper part.

Loads

The experimental body of the RC pier is about 310t.

Moreover, superstructure girder weight with the bearings totaled 307t. Additionally, if the two side frame piers and protecting frame are summed together, the weight on the vibration table for loading was about 1030t.

The input acceleration was the observed waves at JR Takatori station in the Hyogoken Nanbu Earthquake as shown in Figure 3. The three direction components (two horizontal directions and one vertical direction) are input simultaneously. Because of the interaction between the soil and structures, the used acceleration is 80% of the original recorded ones.

Blind Analysis

In order to improve the numerical analysis technology to forecast the response and damage behavior of RC structures during earthquakes, the NIED executed a blind analysis contest against the experimental C1-2 pier. The analysis was performed in the following two stages:

Stage 1) Analysis before experiments:

Forecast the damage behavior corresponding to the seismic ground motion (target waves) input to the shake table.

Stage 2) Analysis after experiments:

Forecast the damage behavior with the same model as Stage 1 and analyze methods corresponding to the seismic ground motion (observed waves) input to the shake table.

This paper describes the results of Stage 2. The contest was held in the following two sections:

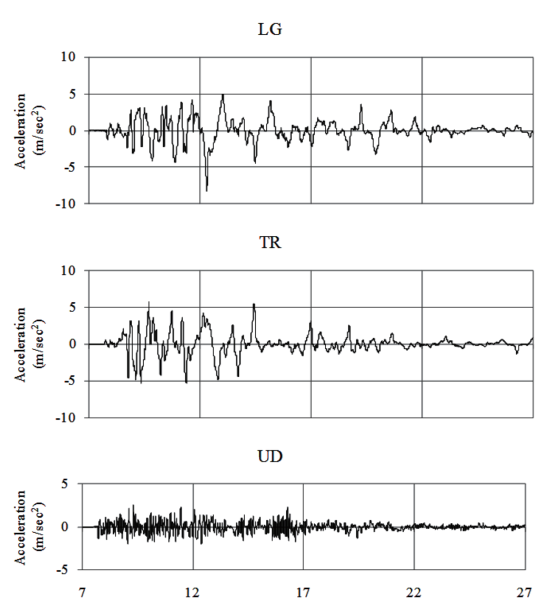
Section A) FEM model analysis section:

Use the finite element method to analyze reinforcement concrete by solid elements.

Section B) Fiber model analysis section:

Use the fiber model to analyze reinforcement concrete by beam elements.

We employed an analytical tool, UC-win/FRAME(3D), which is an all-purpose analysis program for the spatial frames. The program was used to create a fiber model simulation to predict the Section B contest.



▲Figure 3

Among the materials for the analysis, the following five items were opened to the contest participants. They were downloadable from the NIED server.

(1) Shape of structure: Details such as plane drawing, spatial drawing, member sections and bearings

(2) Weight distribution: Details of each member, parts and spindles, and bearing conditions

(3) Material: basic properties and strain-stress relationship of used materials, and concrete composition

(4) Input earthquake waves

(5) Photographs of experimental sample

Analysis Methods

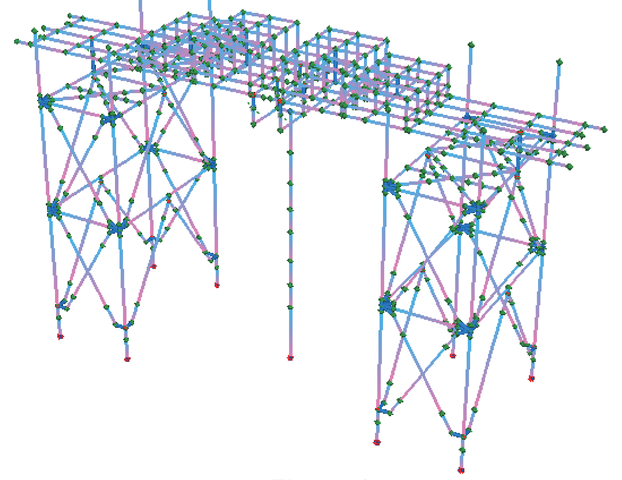
Analysis Conditions

- Analysis tool: UC-win/FRAME(3D) Ver.3.00.02 developed by FORUM8 Co., Ltd.
- Integration method: Newmark β Method (β=1/4)
- Integration interval of time: ΔT=0.005 second
- Input acceleration: Two horizontal directions and vertical direction input simultaneously, observed wave (acceleration) to shake table
- Geometrical consideration: Large displacement analysis (Compatibility Condition: Nonlinear)

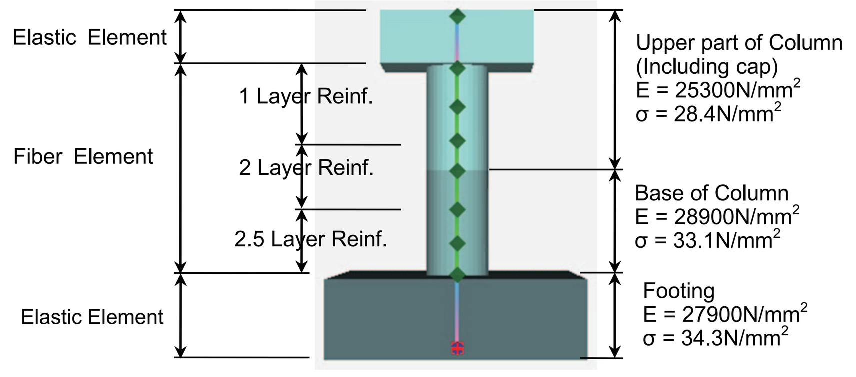
Analysis Model

Figure 4 shows an analytical model with 585 nodes, 735 elastic elements, 8 spring elements, and 6 fiber elements for the pier column. As for the bearing conditions, only the bearing on the top of the pier is fixed, and the side ones are movable. However, the friction of slipping is considered.

The pier where the fiber element is used is described in detail. The cap beam and the footing are assumed to be elastic. As described before, concrete was casted in three phases that included (a) footing, (b) column base part, and (c) column upper part and cap beam. The strength of those materials was published and reproduced. The length of fiber elements is set to the half of section diameter, D=1.8m.The fiber element sections are finely divided and the stress-strain relationship of each cell is defined respectively. Figure 6(c) shows the division chart of the section. Because large stress occurs at a position away from a neutral axis, the cover concrete is divided more finely than the core concrete. As a result, the number of cells in the section becomes 1400.



▲Figure 4

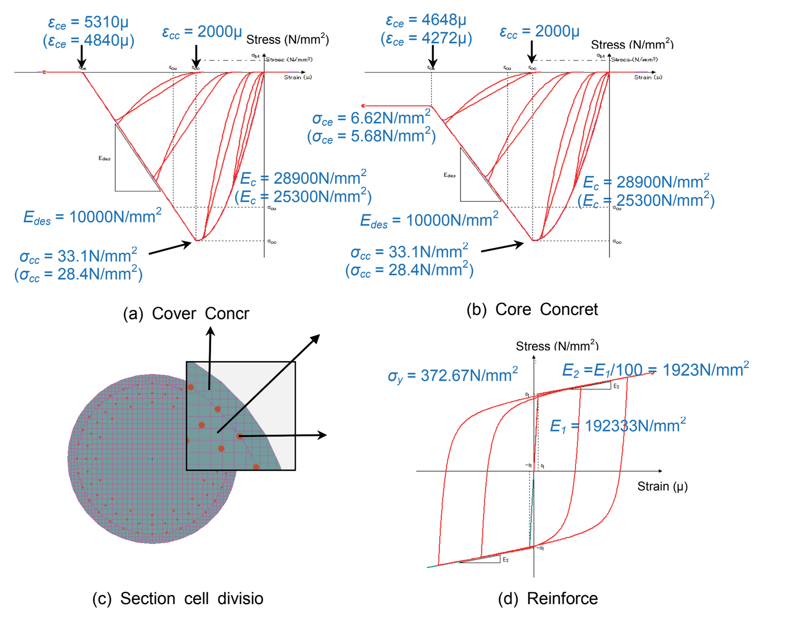


▲Figure 5

The hysteresis of the reinforcement employed is a bilinear type in the bone frame; its inner loop is the modified Menegotto-Pinto model.

Concrete hysteresis is defined differently for cover concrete and core concrete.

The numerical value in Figures 6(a), (b) and (d) is concrete of pier base parts and the value in brackets is a parameter for the upper part concrete.



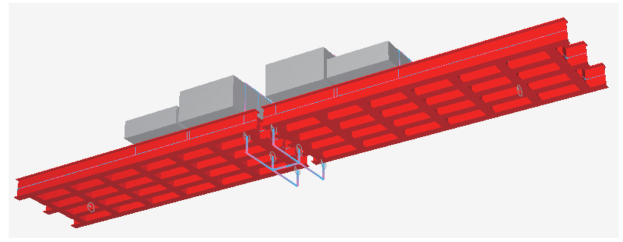
▲Figure 6

Three kinds of springs that modeled bearings were defined as shown in the Figure 7. Fixed bearings (represented as circles) were set up at the top of the pier, and falling-prevention bearings (represented as triangles) were set on both sides of the circles in the transverse direction. Movable bearings (represented as rectangles) by spring elements were set to consider the friction on the sides in the longitudinal direction.

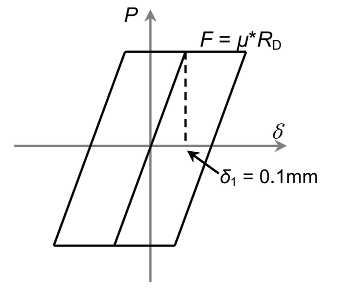
The damping matrix was taken as the proportion type depending on elements. The viscous damping constant of each element was set as follows.

- Girder: 2%
- Concrete (elastic): 5%
- Rigid member and bearing: 0%

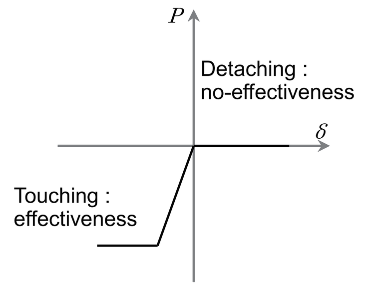
Moreover, the viscous damping constant for the fiber element used for the pier column was assumed as zero so that hysteresis damping only was considered automatically.



▲Figure 7



▲Figure 8



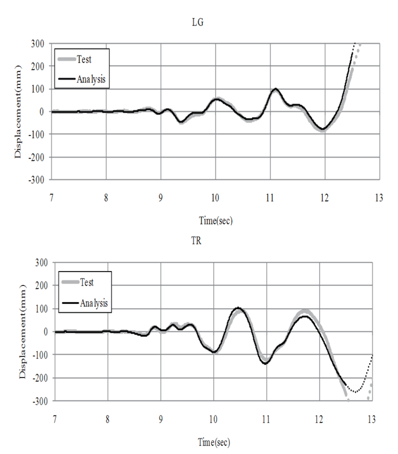
▲Figure 9

Comparison of Analysis and Experiment

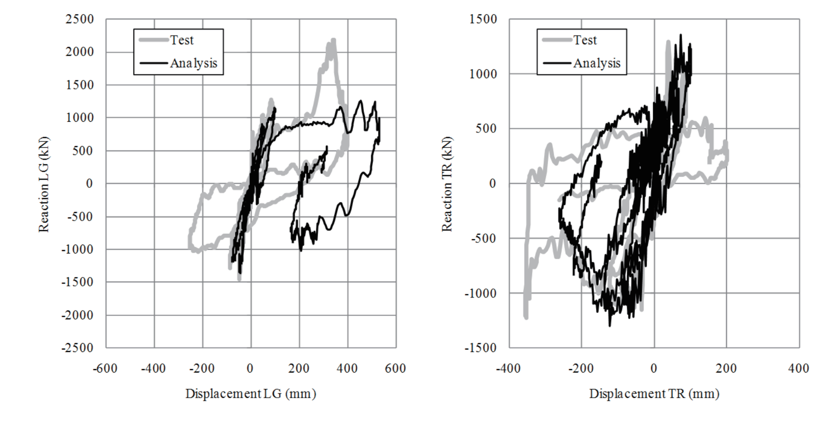
Displacement Histories of Pier Top

Figure 10 shows the displacement history of the top of the pier. It can be concluded that both amplitudes and the period of the experiment could be reproduced well.

The relation between the load and the displacement generated at the fixed bearing position on a central pier is shown in Figure11. The outcome of the experiment and the analytical results correspond within about 200mm as does the comparison of displacements.



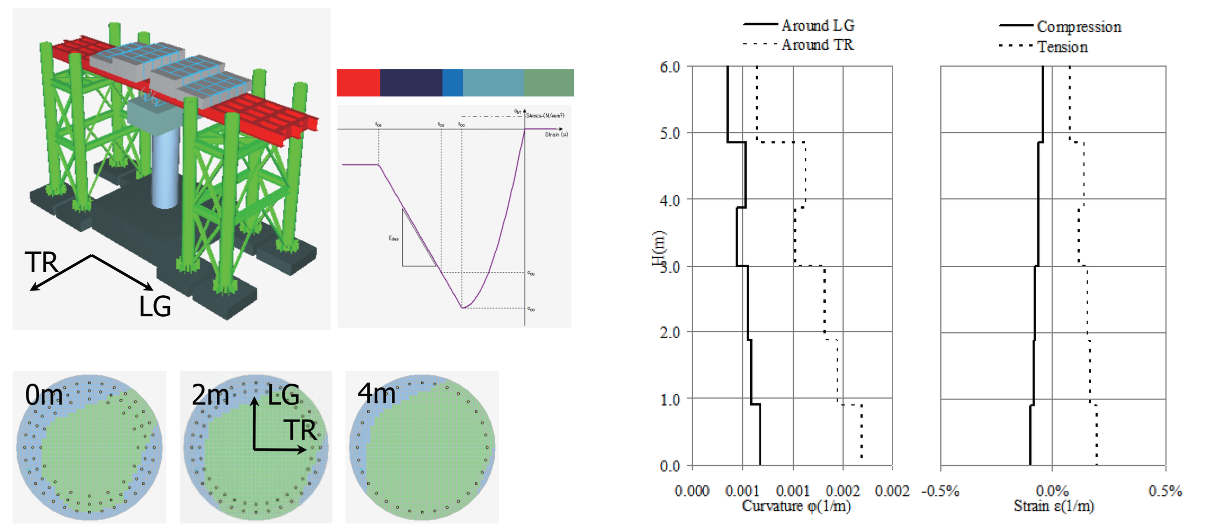
▲Figure 10



▲Figure 11

Distribution of Curvature and Strain Reinforcement Yield of Base Part (T=9.305 Second)

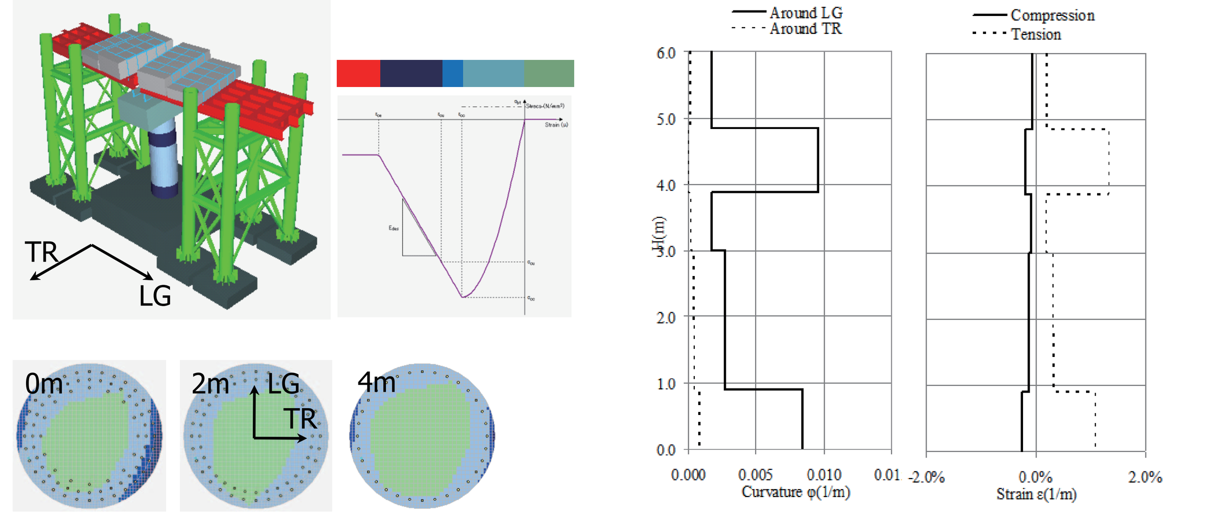
Figure 12 shows the strain distribution and the curvature distribution at 9.305 seconds, the time of main reinforcement yielding. The curvature of cut-off part at the upper part of the column (H=3.9m) becomes somewhat larger but the maximum curvature appears at the base of the pier because of locating in the elastic range.



▲Figure 12

Concrete Ultimate of Cut-off Part (T=10.895 Second)

It is the step that the ultimate state occurred, the curvature and strain at the H=3.9m experienced the maximum. As shown in Figure 13, it can be concluded that the damage at the cut-off part had occurred in the experiment.



▲Figure 13

Conclusions

- Response analysis of time history was executed for the real large experiment body which has a cut-off reinforcement. A comparison between the experiment and analysis was conducted.
- The displacements and forces in the experiment were reproduced very well in macro level of the model analysis.
- A verification on the micro level is planned. This will examine more detailed results between the experiment and analysis according to the materials available from the NIED.

Acknowledgments

We very much appreciate the NIED' s permission to publish materials related to the C1-2 experiment.

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