NONLINEAR RESPONSE ANALYSIS AND DAMAGE EVALUATION OF A CITY OFFICE IN KORIYAMA CITY

Pinar INCI KOCAK* **Supervisor: Toshikazu KABEYASAWA****
MEE12616

ABSTRACT

The focus of this research is a nonlinear response analysis of how a city office in Koriyama city was affected by The Great East Japan Earthquake, 2011. Nonlinear pushover and dynamic analysis are performed in two different analytical models, designated first and second. The first structural model only takes into consideration column flexural capacity without regards to deterioration in post-peak behavior, and the second structural model takes account of shear capacity with regards to post-peak behavior. Using these analyses, a damage evaluation is conducted and compared to the post-earthquake damage report prepared by a research team from the Building Research Institute (BRI) and National Institute for Land and Infrastructure Management (NILIM). Through the use of these comparison results, analytical models commonly used in literature are judged in terms of their adequacy in representing the dynamic response of structures. In addition, there is a critical assessment of the seismic evaluation process used in the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001.

Keywords: Flexural, Shear, Post-peak, Seismic, Nonlinear response

1. INTRODUCTION

On March 11, 2011, the Great East Japan Earthquake (Mw 9.0) hit the north-east of Japan at 14:46 JST (5:46GMT). The earthquake was a massive tsunami caused by the seismic shock, which resulted in the death of more than ten thousand people and a huge number of buildings being completely or partially destroyed or washed-away.

In order to learn from this terrible disaster and contribute to the improvement of disaster mitigation measures against earthquakes and tsunami, both the National Institute for Land and Infrastructure Management (NILIM) and the Building Research Institute (BRI) sent research teams to the affected areas and carried out a comprehensive survey on a variety of building types (BRI research paper, 2011). One of these buildings, a city office in Koriyama city damaged by ground motion was examined and a detailed damage report prepared. In this study, a nonlinear response analysis and damage evaluation of a city office in Koriyama city has been performed, taking into consideration the damage report on the structure.

2. DESCRIPTION OF A CITY OFFICE IN KORIYAMA CITY

A city office in Koriyama city, Fukushima, Japan, a three-storey building, was constructed in 1970 with an RC frame-wall system (Figure 1).

* Istanbul Technical University, Structural and Earthquake Engineering Laboratory, Istanbul, Turkey
**National Institute for Land and Infrastructure Management, Tsukuba, Japan

Figure 1. General view of a city office in Koriyama city

The column application can be seen in Figure 2. As seen in Figure 1, the building extends two spans transversally (y direction) and nine spans longitudinally (x direction). RC walls are located transversally. All columns have a 50x50cm cross-section with different number of longitudinal rebar. Concrete compressive strength was determined as 18.73, 12.60 and 11.19 for the first, second and third storey, respectively and yield strength of plain reinforcing bar is 294 MPa.

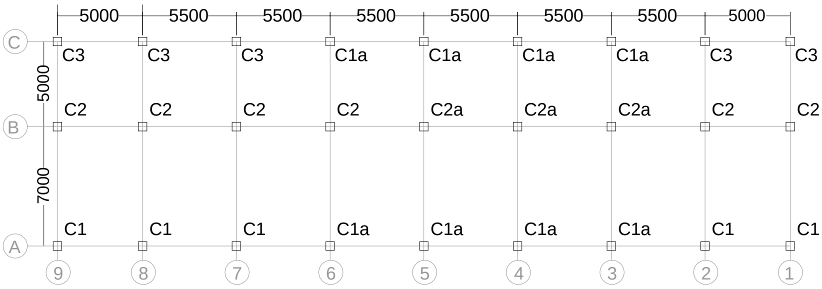

Figure 2. Column application plan of the structure (units in mm)

The strong motion data accumulated by FKS018 station located in Koriyama, Fukushima Prefecture, is used for nonlinear analysis of a city office in Koriyama located about 1.2km from the K-Net Station recorded maximum acceleration as 746.1cm/s/s, 1069.3cm/s/s and 457.4cm/s/s in N-S, E-W and U-D directions, respectively.

The Joint Survey Team investigated the city office in accordance with the damage criteria determined. Medium and heavy damage was found to the first storey columns as results of the existence of spandrel wall top and bottom of the beams (Figure 3).

Figure 3. Shear failure of the first storey columns

3. ANALYTICAL STUDY

Actual structure is modeled for the members with or without consideration of strength deterioration (the first and second structural model, respectively) and nonlinear pushover and dynamic analysis are performed using the CANNY structural analysis software. Performance levels of the structural members are judged according to the Japanese Seismic Evaluation Standard for Existing RC Buildings. In order to determine the member models, such well-known methods are performed in nonlinear analysis and their applicability are tested, taking into account the comparison of the performance level of members determined in analytical study and actual damage report.

3.1. Modeling of Structural Members

A modified version of a one-component element (Giberson, 1967) is used to represent the beams whose force-deformation relationship is linear elastic (Figure 4). One-component model is also proposed for the column members to represent

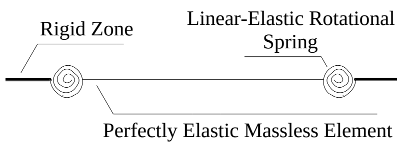

Figure 4. Member model for beams

force-deformation relationship of columns with or without consideration of strength deterioration in the first and second structural model, respectively. Axial deformation is considered in two models. An important point is that nonlinear rotational spring governed by shear deterioration is introduced instead of a rotational spring or shear spring assigned serially in the second structural model (Figure 5).

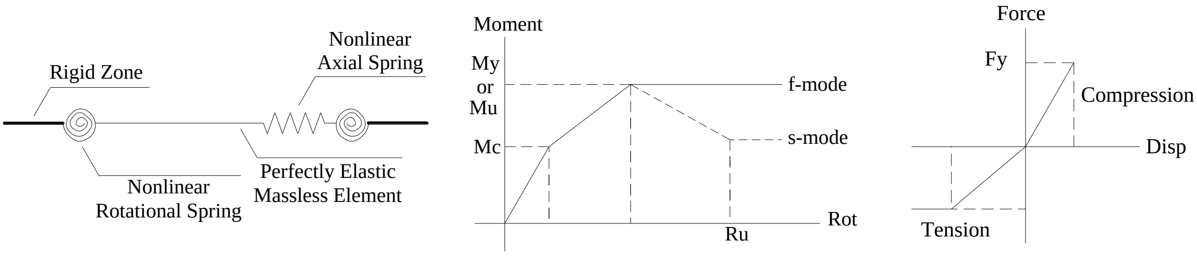

Figure 5. Member model for columns and force-deformation relationships

In the first structural model, cracking moment *Mc* and yield moment *My* is calculated by using Eqs. (1) and (2). In the second structural model, the ultimate shear strength of the columns are determined as the smaller value of the ultimate shear force (*Vu*) and corresponding shear force (*Vy*) of the yielding moment (*My*). Then, the post-peak behavior of the columns is determined as s-mode (shear failure) columns or f-mode (flexural failure) columns according to the ultimate shear strength capacity. For s-mode columns, ultimate shear force (*Vy*) is assigned (Eqs (3) and (4)). Skeleton curves of the nonlinear springs can be seen in Figure 5. After determination of column failure mode as s-mode or f-mode, to detect the stiffness properties of the rotational spring, the formulas introduced by Yoshimura (2005) are used for s-mode columns Eq. (5) and f-mode columns Eq. (6). Hysteresis curves of the s-mode columns can be seen in Figure 6. The axial nonlinear deformation relationship is defined by using Eqs. (7) and (8) for both structural models.

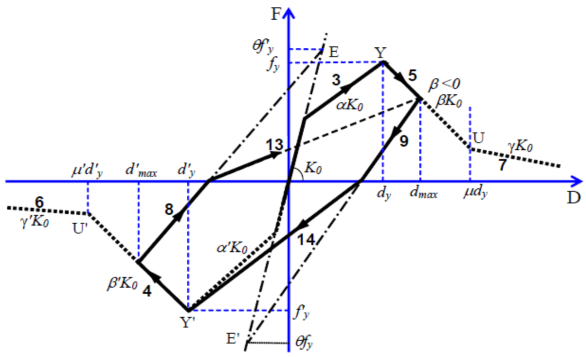

Figure 6. Hysteresis curves of s-mode columns

Three vertical line element model (TVLEM) determined in Kabeyasawa *et al.*, is assigned for the transverse walls of the structure which has linear-elastic stiffness characteristics.

$$Mc = 0.56\sqrt{\sigma_B}.Z_e + \frac{N.D}{6} \tag{1}$$

$$M_y = 0.8.a_t.\sigma_y.D + 0.5\,N.D.(1 - \frac{N}{b.D.F_c}) \tag{2}$$

$$Vu = \left(\frac{0.068\,{p_t}^{0.23}(180+Fc)}{\frac{M}{QD}+0.12} + 0.85\sqrt{p_{ws}.\sigma_{wy}} + 0.1\,\sigma_0\right).b.j \tag{3}$$

$$Vy = \frac{My}{(h_{column})/2} \tag{4}$$

$$Ru = \frac{ds}{l} = 62.2\,p_w - 51.9\eta + 6.07p_s - 9.91 \geq 1.5 \tag{5}$$

$$Ru = \frac{du}{l} = 28\,p_w - 42.3\eta - 8.65p_s + 20.6 \geq 1.5 \tag{6.a}$$

$$Pu = Vy - 0.075\,Py.(Ru - 2) \tag{6.b}$$

$$F_y = A.F_c \tag{7}$$

$$F_y' = A_s.\sigma_s \tag{8}$$

where σ_B is concrete compressive strength, *Ze* is section modulus, *N* is the axial force of the column, *D* is the cross-sectional height of the column and *Fc* is the concrete compressive strength (like σ_B). p_{ws} is the transverse bar ratio, σ_0 is average axial stress over the entire wall cross-sectional area and *j* is a geometrical parameter. *Ru* is the collapse drift, *ds* is the displacement at the collapse for s-mode columns, *l* is the column height, *pw* is the transverse reinforcement ratio (%), q_s is the longitudinal reinforcement ratio (%) and *du* is the displacement value for collapse for f-mode columns. F_y, F_y' is the axial force in compression and in tension, respectively, and *As* the total cross-sectional area of longitudinal reinforcement.

3.2. Structural analysis

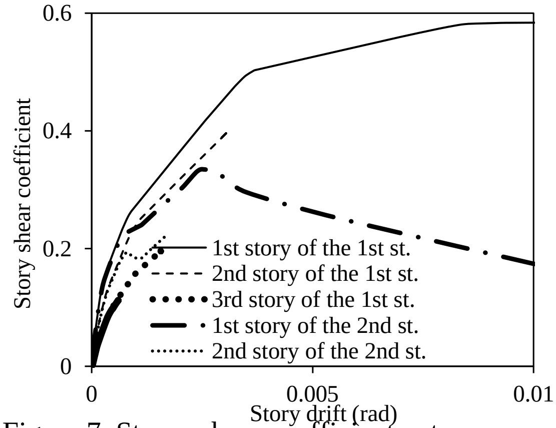

Figure 7. Storey shear coefficient - storey drift ratio relationships of nonlinear pushover analysis

The storey shear coefficient - storey drift ratio relationships obtained by the nonlinear pushover and dynamic analyses for both the first and second structural models are shown in Figure 7 and 8. As distinct from actual structural damage in Figure 8, the response of the second storey seems greater than that of the first in the second structural model. The seismic performances of the structural models follow the performance criteria laid out by the Standard for Seismic Evaluation of Existing RC Buildings. The demand spectrum of the Great East Japan Earthquake (*Sa*-*Sd*) is obtained through ViewWave software. The displacement demand (*D1*) is determined and the corresponding displacement demand (*D2*) of the equivalent damping (*he*) of the building is obtained where the capacity diagram and demand spectrum intersect (Figure 9). Corresponding demand points of the 1st storey columns are determined. The deterioration factor (η) is the ratio of residual dissipation capacity (E_r) to total dissipated energy capacity (E_d+E_r) is calculated (Figure 10). The deterioration factor (η= E_r/(E_d+E_r)) of columns is given in Table 1, which shows the damage level from I to V as determined by analytical calculation. The damage rank of the first storey columns is given in the storey plan (Figure 11). In Figure 11, the first, second, third and the fourth damage rank of a column represent the damage level determined by static and dynamic analysis of the first structural model and static analysis of the second structural model and onsite survey (actual damage), respectively. The damage rank of the columns calculated through static and dynamic analysis conforms to each other in the first structural model. This indicates the reliability of the nonlinear dynamic analysis. However damage ranks determined from the second structural model and actual damage ranks are not exactly the same, especially for inner side columns (axis B); results have

Table 1. Deterioration factor

Damage level	Flexural member	Shear member
I	0.95	0.95
II	0.75	0.6
III	0.5	0.3
IV	0.1	0
V	0	0

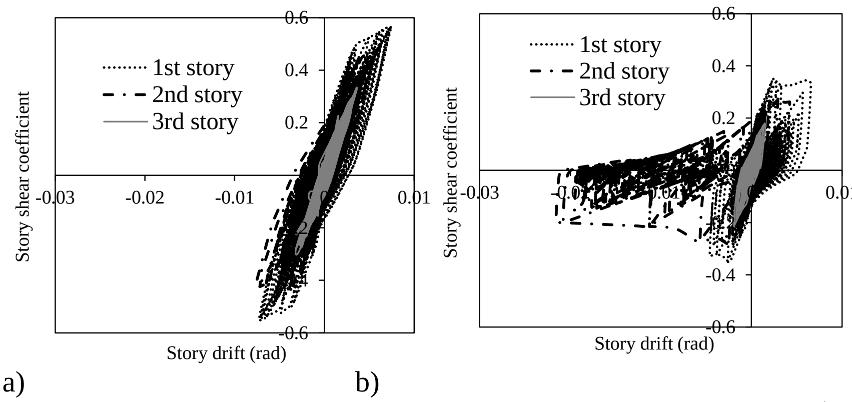

Figure 8. Storey shear coefficient - storey drift ratio relationship a) 1st st. model, b) 2nd st. model

acceptably minor differences in outer side columns (axes A and B). The difference in damage rank results on the B axis is reasonable, considering the possible effects of nonstructural members on the inner axis columns.

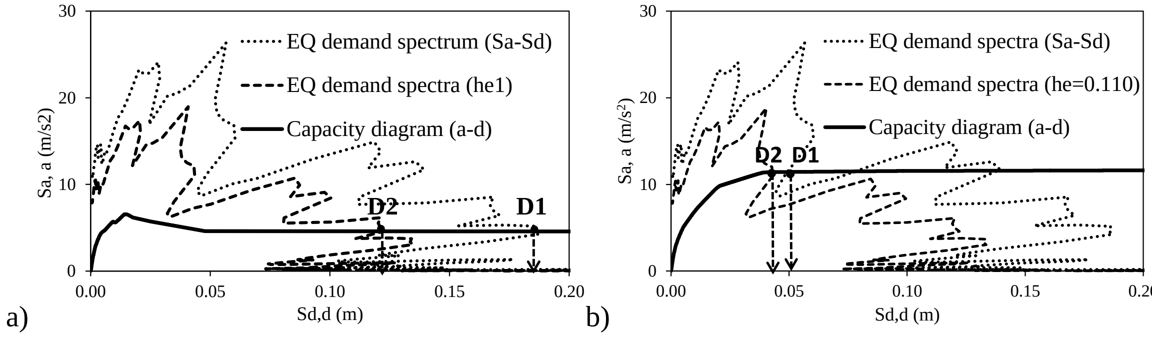

Figure 9. Capacity diagram and demand spectrum relationships a)1st structural model, b) 2nd structural model

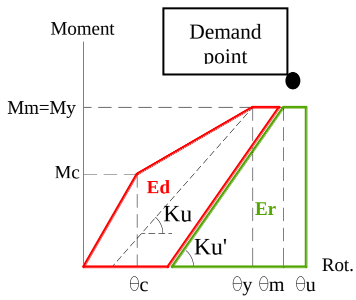

Figure 10. Energy deterioration concept of structural members

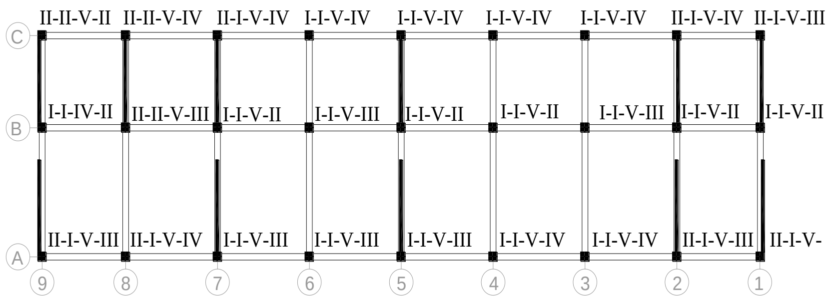

Figure 11. Damage ranks of the first storey columns

The strength index C and the ductility index F are calculated in accordance with the second level screening procedure determined by the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (Figure 12). Good correlation was found between the storey strengths obtained from the second level screening procedure and nonlinear pushover analysis of the second structural model, it should be emphasized that the decreasing storey strength without regard to any post-peak behavior in second level screening procedure is a conservative approximation when compared to the energy consuming capacities.

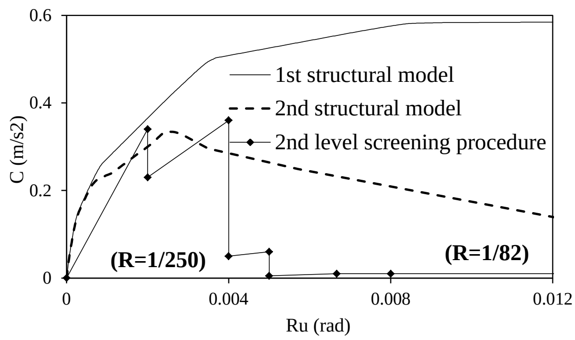

Figure 12. Storey shear-inter-storey displacement relationship for first storey

4. CONCLUSION

A city office in Koriyama city was modeled using CANNY structural analysis software to produce two different models; the first and second structural models. These two structural models were subjected to nonlinear pushover and dynamic analyses, and the seismic structure responses were compared, both to each other and to the actual damage reports. At the same time, the strength index C-ductility index F relationship, a common method to evaluate the seismic capacity of existing RC structures proposed by the Japan Building Disaster Prevention Association, is compared to the analysis results .

- The strength capacity of the second structural model can be increased to match that of the first structural model by decreasing column hoop distance, but it should be kept in mind that only increasing strength is not enough; ductility loss, as a result of strength deterioration in the cross-section of the columns, should also be considered.
- However in this study, the responses of the second structural model have been on the conservative side when compared to the actual damage, it is obvious that a more detailed analysis is needed. Especially, the existence of nonstructural walls, soil-structure interaction and slight cracks in the beams can be modeled to reduce the difference in the damage levels of the second structural model and the actual damage.
- The city office in Koriyama city represents a type of commercial office building often found in Turkey. So, in particular, further study is needed to compare methods of seismic evaluation of existing RC structures in Turkey and Japan to contribute to the reconstruction process led by the Ministry of Environment and Urbanization in Turkey.
- Comparison of the energy absorbing capacities from the under the area of the strength index curves determined from the second structural model and by using the second level screening procedure shows us with clues that the second level screening procedure is more conservative damage estimate method. The energy consuming capacity is still going on in second structural model, when the strength is directly reduced in the second level screening procedure.

ACKNOWLEDGEMENTS

A special acknowledgment is extended to Mr. Mizuo Inukai and Dr. Kashima who helped at various stages of the thesis. City officer in Koriyama city is acknowledged for giving the information about the city office in Koriyama.

REFERENCES

BRI Research Paper (Technical note of National Institute for Land and Infrastructure Management), (2011). Summary of the field survey and research on the ‘2011 off-the-Pacific coast of Tohoku Earthquake’. ISSN 1346-7328, Technical note of NILIM No.647, ISSN 0453-4972, BRI Research Paper No.150.

CANNY for three dimensional nonlinear static and dynamic structural analysis, version C06.

Giberson, M. F., (1967). The Response of Nonlinear Multi-Storey Structures Subjected to Earthquake Excitation, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California, EERL Report.

Strong-motion seismograph networks (K-NET, KiK-net), http://www.kenchiku-bosai.or.jp

Japan Association for Building Disaster Prevention (2001). Standard for Seismic Evaluation of Existing RC Buildings (in Japanese).

Kabeyasawa, T., Shiohara H., Otani S. and Aoyama, H., (1983). Analysis of the Full-scale Seven-storey Reinforced Concrete Test Structure. The Faculty of Engineering, University of Tokyo, Vol. XXXVII, No. 2, pp. 432 - 478.

ViewWave (2007), a viewer for strong motion records, Building Research Institute, Tsukuba, Japan.

Yoshimura, M. and Takaine, Y., (2005). Formulation of post-peak behavior of reinforced concrete columns including collapse drift, Journal of Construction Engineering, Architectural Institute of Japan, No.587, 163-171. (in Japanese).