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SEISMIC RESPONSE OF DETERIORATED RESIDENTIAL RC BUILDINGS IN THE NORTHEASTERN REGION OF THAILAND

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ABSTRACT: This paper presents effect of steel corrosion and concrete degradation on seismic response of residential RC buildings in the Northeastern region of Thailand, originally designed without seismic consideration**.** Two parts of the study are presented, including site investigation to gather information of damages in 10 existing buildings, and nonlinear pushover analysis of deteriorated structure based on damage scenario from the first part. Using procedure and parameters described in ASCE 41 and recent results of seismic hazard assessment, the numerical results revealed that buildings of all material degradation conditions were safe under the Basic Safety Earthquake (BSE). The evaluation of the structures under the Design Earthquake (DE) indicated that the studied building with high deterioration could not withstand the design earthquake, as the most loaded columns would fail in flexure rather than in shear associated with yielding in the beams of the buildings. Ductility of buildings was found to be sensitive to the supposed material degradation.

Keywords: Seismic response, Residential RC building, Material degradation, Pushover analysis

1. INTRODUCTION

At present, seismic events occur more frequently in Thailand and sometimes lead to structural damages in many residential buildings**.** Damage may be severe especially for those buildings that were originally designed in the past without seismic consideration. Majorities of residential buildings in Thailand are mostly low-rise and normally made of reinforced concrete. When these buildings have been used for some time, degradation of structural materials is often observed in the form of steel reinforcement corrosion [1-2], change of bonding between reinforcement and concrete [2-6] including strength degradation of concrete [7,8]. Consideration of material degradation into seismic damage assessment of these structures is important to expose the realistic remaining load resistance and deformation ability of these structures.

These issues are essential, especially for existing buildings deemed in the past to be free from seismic action but now revealed to locate in seismic hazard zones, which is the case of Northeastern region of Thailand [9,10]. Recent literature reviews have suggested that study of seismic induced damages of deteriorated residential buildings in this region of Thailand is lacking.

In this paper, effect of rebar corrosion and concrete deterioration on seismic response of a reinforced concrete residential building was studied**.** The sample building is of three-story three-unit type, and was not originally designed to resist seismic loading. The paper consists of two parts, the first part involves a site investigation of selected existing buildings in Khon Kaen Thailand, to explore possible material degradation scenarios for further use in modelling response of the sample building. In the second part, the updated acceleration response spectrum for Northeastern Thailand [9,10] was used as an input in the present study. Pushover analyses via the structural analysis software SAP2000 [11], including the capacity curves and failure mechanisms of the building, were used to investigate performance of the building with different levels of material degradation. Damage assessment procedure conducted in this study was based on ASCE 41 [12]. The objective was to understand possible building damages for necessary public preparation.

2. MATERIAL DEGRADATION

Degradation of concrete and corrosion of rebars can affect structural behavior of reinforced concrete buildings**.** The concrete strength is decreased over time**.** Some cracks may occur**.** According to the previous research [1,13], change in mechanical properties of rebar can lead to decrease of ultimate elongation and ductility, even when the area of rebar is slightly reduced**.** Once air penetrates concrete, it activates carbonation process and correspondingly affects debonding between steel rebar and concrete, resulting in corresponding corrosion of the rebar and further cracking in the concrete. With material degradation, load resistance of reinforced concrete members is decreased [14].

Steel corrosion has some influences on mechanical properties of concrete as well. The oxide layer around corroded steel circumference creates tensile stress in the concrete, which may be beyond the concrete resistance. The research works for this subject are still ongoing [2-6]. Effect of corroded steel on interaction between steel rebar and concrete is still a complex issue and is dependent on number of involved parameters, such as rebar position, concrete quality, steel quality, outer concrete cracking, corrosion level, and location of structure. In this paper, corrosion was treated by area reduction of the rebar due to corrosion [7,8]. The stress throughout yielding process of the rebar was treated in a similar way. Reduction of compressive strength and elastic modulus of the concrete were adopted to mimic concrete degradation based on site investigation report.

3. SITE INVESTIGATION

To gather information about some typical damages, 10 existing residential building structures were investigated. The buildings were more than 20 years of age, and were not at that time designed to resist any seismic load. Most of the existing damage obtained from the site investigation involved with rebar corrosion. Having rust on surface of the rebar, cross sectional area of the rebar was reduced, adding pressure on the rebar-concrete interface, and leading to cracking and spalling of the surrounding concrete. This type of damage was found in some columns, especially those exposed to water and air, such as exterior ground and exterior first floor columns as shown in Fig.1.

Fig.1 Examples of the damage found in the exterior first floor columns.

To quantify level of steel damage, rust on the rebar surface was removed and diameters of the remaining rebar were measured to determine the effective remaining cross-sectional area. It was found in the exterior first floor columns, shown in Fig.2, that the bars lost 5.14 to 26.86% of their original area. Compressive strengths of the concrete, in its present condition, were also investigated using a rebound hammer test at totally 12 random positions. The concrete strengths were found to be 18.4-36.0% decreased from the design strengths. The investigation showed that there were correlations between the degree of observed steel corrosion and strength degradation of concrete.

Information from the site investigation was used to set up material degradation scenarios, the observed initial damages existed mainly in the exterior ground columns and the exterior first floor columns along perimeter of the buildings, as shown in Fig.2. In this study, five different levels of material degradation defined in Table 1 were chosen to resemble the observed data of these existing damages. The surveyed buildings are based on residential RC buildings in the area of Khon Kaen University, Thailand.

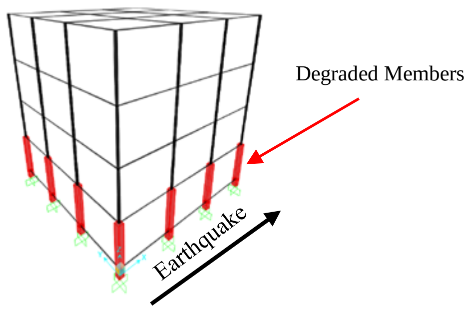

Fig.2 Position of the damaged RC columns.

Table 1 Material degradation levels used in this study.

Material Degradation	Percent reduction of concrete strength (%)	Percent reduction of rebar area (%)
No degradation	0	0
COR0-20	0	20
COR20-20	20	20
COR30-30	30	30
COR40-40	40	40

4. SEISMIC HAZARD IN THE NORTHEAST OF THAILAND

In this study, spectral acceleration (Sa) for Northeastern part of Thailand, recently proposed [9,10] was employed. The adopted seismic hazard maps were as shown in Fig.3 and Fig.4, illustrating contours of spectral acceleration for a 2% probability exceedance in 50 years corresponding to the Maximum Considered Earthquake (MCE). These accelerations were stronger than values suggested in the current Thai seismic design standard DPT1301/1302-61 [15]. Adopted location for this study was Bueng Kan province, situated at the upper right corner of the maps (see Fig.3 and Fig.4), where the maximum acceleration in this region was suggested. Fig.3 and Fig.4 show the maximum acceleration of 0.70g and 0.20g at the natural periods of structure of 0.2 sec and 1 sec, respectively.

To evaluate seismic damage of the buildings, two seismic intensity levels were investigated, including the Basic Safety Earthquake (BSE) for which its intensity level is one third of the Maximum Considered Earthquake, and the Design Earthquake (DE) for which its intensity level is two thirds of the Maximum Considered Earthquake. From both seismic intensity levels, two spectral accelerations were used in the response spectrum analysis following the Thai standards DPT1301/1302-61 [15] and DPT1303-57 [16] as shown in Fig.5. It was assumed that the building was placed on normal soil layers.

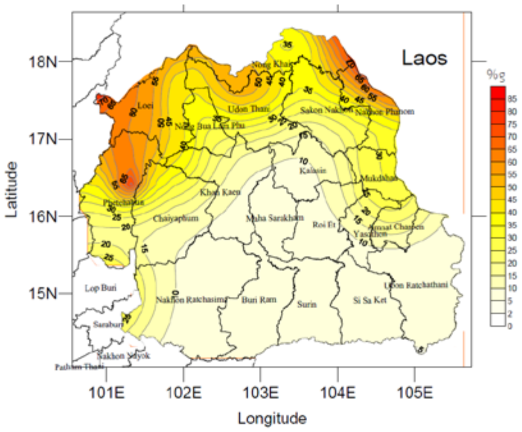

Fig.3 Seismic hazard map of the Northeast Thailand corresponding to 2% probability of exceedance in 50 years, Sa for structural period at 0.2 sec. [10]

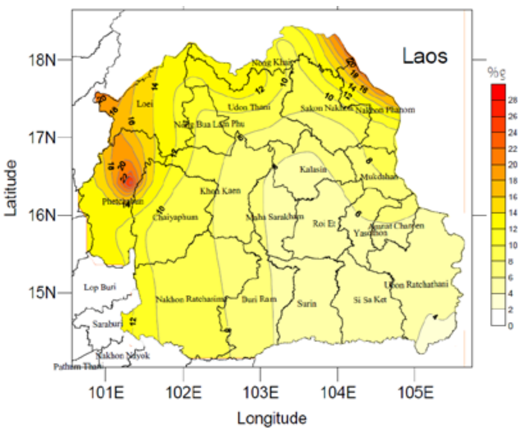

Fig.4 Seismic hazard map of the Northeast Thailand corresponding to a 2% probability of exceedance in 50 years, Sa for structural period at 1.0 sec. [10]

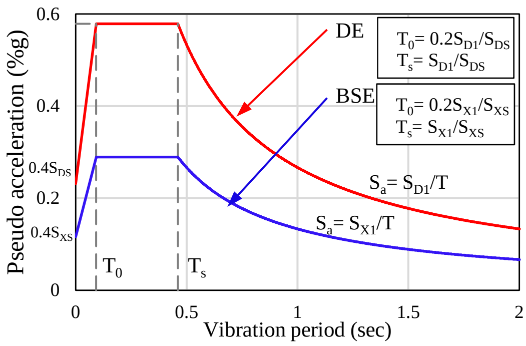

Fig.5 The adopted acceleration response spectrum for the reinforced concrete building.

5. MODELLING OF BUILDING RESPONSE

5.1 Building Structure

The sample of building structure used in the numerical study was a three-story and three-unit residential RC building that is one of typical buildings found in all regions of Thailand. The reinforced concrete building was totally 12 m wide, 12 m long and 9.5 m high, as shown in Fig.6.

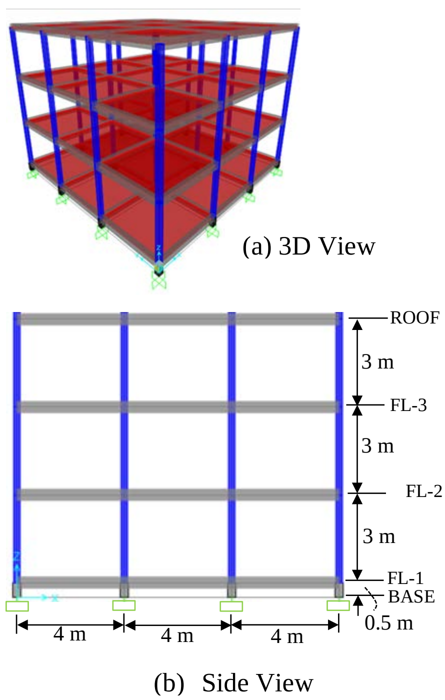

Fig.6 The building structure.

Modelling of the building was based on nonlinear frame analysis by the structural analysis software SAP2000 [11]. Effective sectional properties were applied, reduction factors of 0.35 and 0.70 were multiplied with the original moment of inertia of cross section for beams and columns, respectively. Footings of the building were modelled as fixed supports. The superimposed dead load was 120 kg/m^2 and the live load was 300 kg/m^2, allowing for commercial option for this type of buildings. The damping ratio of 0.05 was employed [15]. Based on the material properties in Table 2, the beam sections and the column sections are summarized in Table 3 and Table 4. These sections were designed to satisfy only gravity and wind loading on the structure, with no seismic consideration.

In this study, plastic hinge models were used for nonlinear analysis of moment-resisting frame following to ASCE41-13 [12]. For the beams consisting of equally spaced stirrups, the plastic hinge model corresponding to non-conforming transverse reinforcement (NC) was applied. For the columns consisting of closed hoops with 90-degree hooks and shear demand/capacity ratio was less than 1, the plastic hinge model was based on flexure-shear failure. The effect of infilled walls was not considered.

Table 2 Material parameters of the RC members.

Concrete	$f_c^{'}$ = 210 kg/cm^2	E_c = 219,109 kg/cm^2
Longitudinal reinforcement	f_y = 4,000 kg/cm^2	E_s = 2.04x10^6 kg/cm^2
Transverse reinforcement	f_{vy} = 2,400 kg/cm^2	E_s = 2.04x10^6 kg/cm^2

Table 3 Details of the beam sections.

Beam	Size (cm.)	Steel reinforcement		
		Top	Bottom	Transverse
Support	20x40	3-DB16	2-DB16	RB6@ 20
Mid span	20x40	2-DB16	3-DB16	RB6@ 20

Table 4 Details of the column sections.

Column	Size (cm.)	Steel reinforcement	
		Longitudinal	Transverse
Ground	25x25	8-DB16mm	RB6@ 20 cm.
Floor 1,2,3	20x20	6-DB16mm	RB6@ 20 cm.

5.2 Lumped Plasticity Frame Model

Nonlinear analysis of the building was based on the lumped plasticity model, in which the localized inelastic behavior was defined at both ends of each RC member, while the rest of the member behaves elastically. The plastic hinge properties and the corresponding performance levels were specified to control flexural resistance of the member. To detect shear failure, the shear demand/capacity ratio was calculated during the analysis. The moment-rotation relation of the plastic hinge was defined by 5 points, shown in Fig. 7. Based on ASCE41 [12], the linear elastic behavior from A to B until it reaches B, the yielding capacity of the section. The stiffness is decreased from B to C, and shows a sharp drop at C. At D, there still exists some force residuals. When the deformation goes further to E, the plastic hinge shows no force resistance anymore. The performance of the structure can be divided into three levels including immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

In the frame analysis, two types of plastic hinges were used, as indicated in Fig.8. For the columns, the so-called P-M hinges were applied to include effect of axial load. For the beams, the flexure hinges (M3 hinge) were used to include only effect of bending moment. The hinges were located at the middle of plastic hinge length L_p defined as $L_p = 0.0008L + 0.0022 d_b f_y$ where L is the distance from a critical section to the point of contraflexure in cm, d_b is the diameter of longitudinal reinforcement in cm and f_y is the yield stress of longitudinal reinforcement in kg/cm^2.

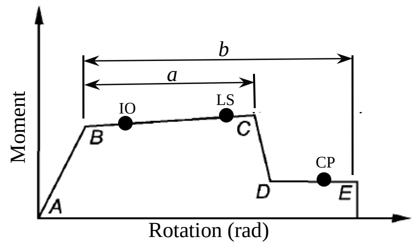

Fig.7 Moment-rotation relation and performance levels defined in the plastic hinge per ASCE41-13 [12].

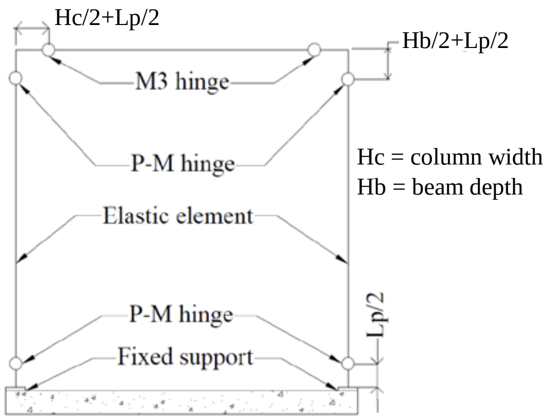

Fig.8 Locations and types of the plastic hinges for the frame analysis.

5.3 Response Spectrum Analysis

Target displacements for the nonlinear pushover analysis were determined by the linear response spectrum analysis, under two seismic intensity levels i.e. the Basic Safety Earthquake (BSE) and the Design Earthquake (DE), as aforementioned in Section 4. To provide an example, the first four mode shapes, corresponding natural periods and mass participation ratios (Γ_n) of the non-degraded building are shown in Fig. 9. The target displacements at the top of buildings are as noted in Table 5. Due to material degradation, structural stiffness decreased, thus increasing the natural period and lowering the acceleration response of the building for the first mode shape.

Table 5 Target displacements for the pushover analysis, corresponding to the first mode.

Degradation level	Natural period (sec.)	S_a (g) of 1st mode shape		Target displacement (cm)	
		BSE	DE	BSE	DE
No degradation	1.39	0.096	0.191	5.89	11.77
COR0-20	1.39	0.096	0.191	5.89	11.77
COR20-20	1.42	0.094	0.188	5.96	11.92
COR30-30	1.43	0.093	0.186	6.01	12.02
COR40-40	1.45	0.092	0.183	6.07	12.13

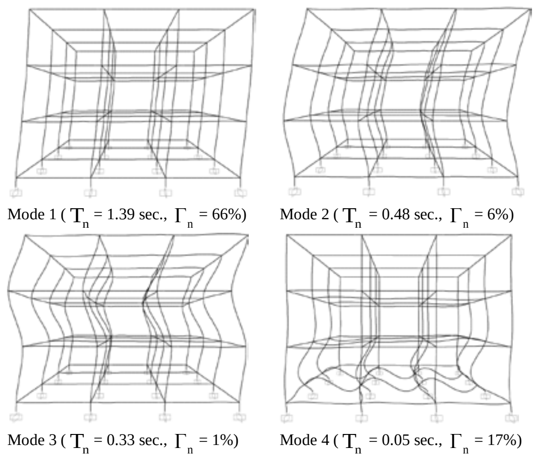

Fig.9 The first 4 mode shapes and the associated dynamic properties for the non-degraded building.

5.4 Moment-Rotation of RC columns

According to ASCE 41-13 [12], parameters affecting sectional plastic rotation include axial load P, shear force V, compressive strength of concrete f_c', and area of transverse steel in closed hoops with 90-degree hooks. It was found in this study that decrease in concrete strength and area of transverse steel also led to decrease in flexural resistance and ultimate plastic rotation of the reinforced concrete members. Fig.10 shows the moment-rotation relation for an exterior first floor column under different levels of material degradation. It was found that the flexural resistances were 15.5%, 16.7%, 24.7% and 32.5% decreased in the members with the damage levels COR0-20, COR20-20, COR30-30 and COR40-40, respectively. Table 6 summarizes the transverse reinforcement ratios and the modeling parameters (cf. Fig.7) for the moment-rotation relation of the first-floor corner column.

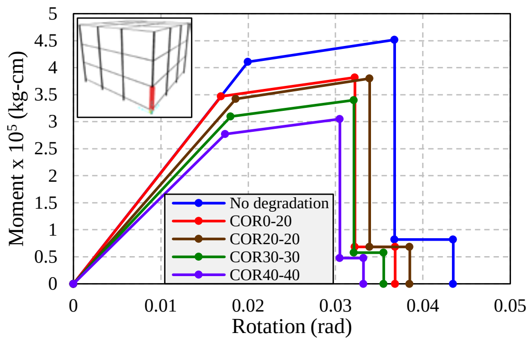

Fig.10 Moment-rotation relation in plastic hinges of the first-floor column under different levels of material degradation.

Table 6 The transverse reinforcement ratios and the modeling parameters for the moment-rotation relation of the first-floor corner column.

Material Degradation	$\rho = \dfrac{A_v}{b_w s}$	Modeling Parameters (Radian)	
		a	b
No degradation	0.0018	0.167	0.0233
COR0-20	0.0014	0.153	0.0199
COR20-20	0.0014	0.153	0.0199
COR30-30	0.0012	0.141	0.0175
COR40-40	0.0011	0.131	0.0158

6. PUSHOVER ANALYSIS RESULTS

In the pushover analysis, the gravity load was initially applied on the building before the building was laterally pushed corresponding to the first mode shape until it reached the target displacement. The analysis results revealed that, with higher levels of material degradation, the ultimate base shear resistance became lower. Relationships between the base shear and the controlled displacement at the top of buildings are shown in Fig. 11. For all material degradation scenarios, the structural behaviors remained in the linear elastic regime under the BSE target displacement. No damage was found in beams and columns of the buildings. Upon further pushing to the DE target displacement, the building responses of each case entered the nonlinear regime. It should be noted that level of material degradation affected formation of the plastic hinges in the beams and the columns. With low level of material degradation, forming of the plastic hinges in the beams occurred earlier than in the columns. With higher level of material degradation, the first plastic hinges were formed in the exterior columns instead.

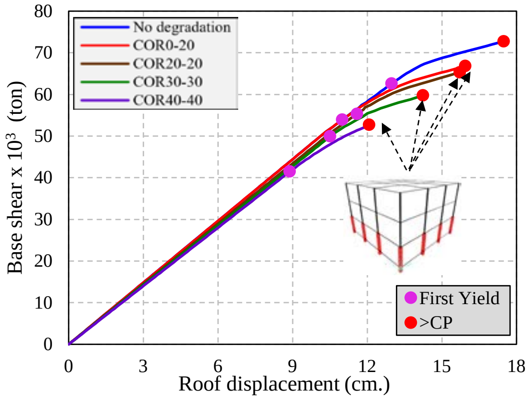

Fig.11 Capacity curve of the buildings for the first-floor columns under various damage levels.

Figs. 12-14 show the responses of the buildings with no degradation and with the low level of material degradation (COR0-20 and COR20-20). The yielding was observed in the second-floor beams first, then in the first-floor columns. From Figs. 15-16, for the building of higher levels of material degradation (COR30-30 and COR40-40), yielding of the first-floor columns was instead triggered due to the significant reduction of the sectional capacity of the deteriorated columns. For the COR40-40 building, failure in the first-floor columns was detected by reaching the limit of collapse prevention (CP), at controlled displacement of 12.08 cm. For all the studied buildings, shear transfers across the beam-column joints were found to exceed the shear joint capacity at the positions shown in Fig.17. The final plastic hinge formation in the buildings under the Design Earthquake are shown in Fig.18.

As shown in the capacity curves, descriptions of the critical points marked in Figs. 12-16 are described in Table 7. Table 8 summarizes the effect of material degradation on the structural ductility (μ) of the studied buildings. Structural retrofit of these buildings should be considered to allow for higher deformability of the building.

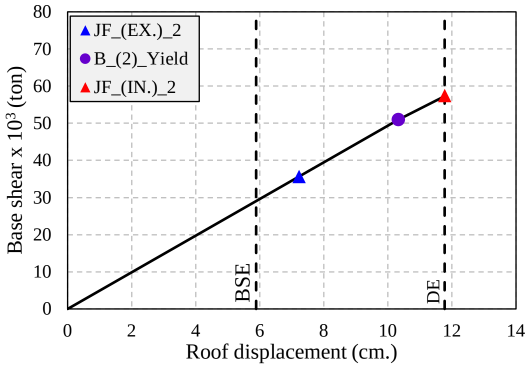

Fig.12 Capacity curve of the building for the first-floor columns with no material degradation.

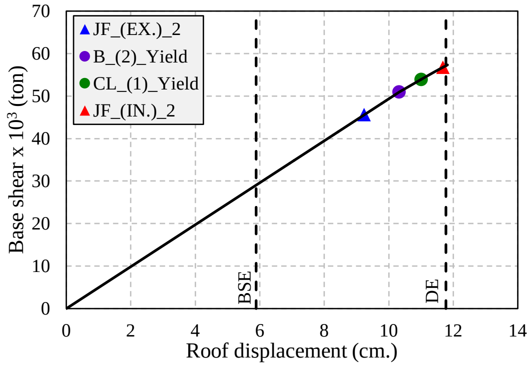

Fig.13 Capacity curve of the building for the first-floor columns under the damage level COR0-20.

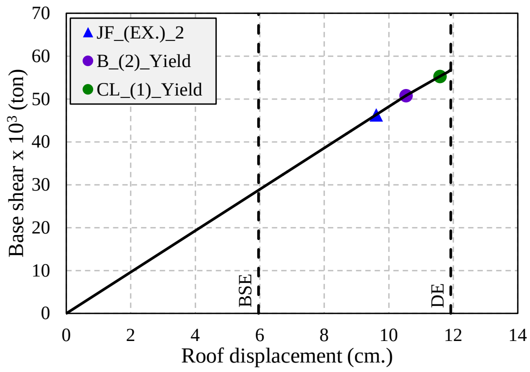

Fig.14 Capacity curve of the building for the first-floor columns under the damage level COR20-20.

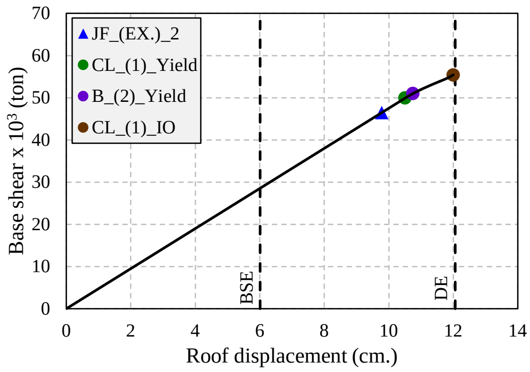

Fig.15 Capacity curve of the building for the first-floor columns under the damage level COR30-30.

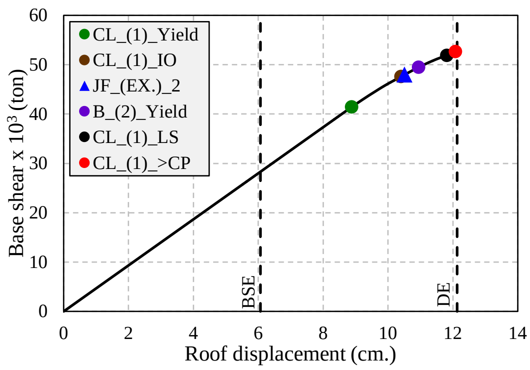

Fig.16 Capacity curve of the building for the first-floor columns under the damage level COR40-40.

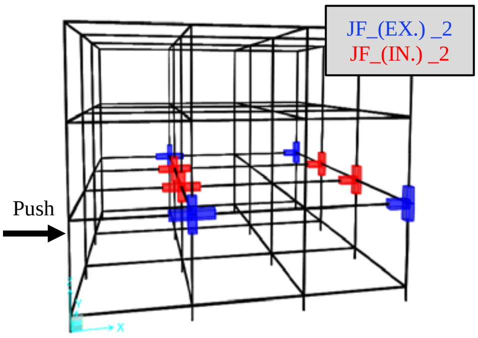

Fig.17 The observed damage in beam-column joints.

As the shear effect was not included in the plastic hinge information, shear demand capacity ratios (shear DCR) of the columns were checked. It was found that beams and columns of the studied buildings were not affected by shear damage. As shown in Fig. 19, the maximum inter-story drifts were observed at the second floor of the buildings and was affected by levels of material degradation.

Table 7 Descriptions of the critical points marked in Figs.12-16.

Status	Descriptions
B_(2)_Yield	The second-floor beam reached its yield condition.
CL_(1)_Yield, IO, LS	The first-floor column reached its yield condition, limit of the IO, LS criteria respectively.
CL_(1)_>CP	The first-floor column exceeded limit of the CP criteria.
JF_(IN,EX.)_2	The second floor, interior or exterior beam- column joint failed in shear.

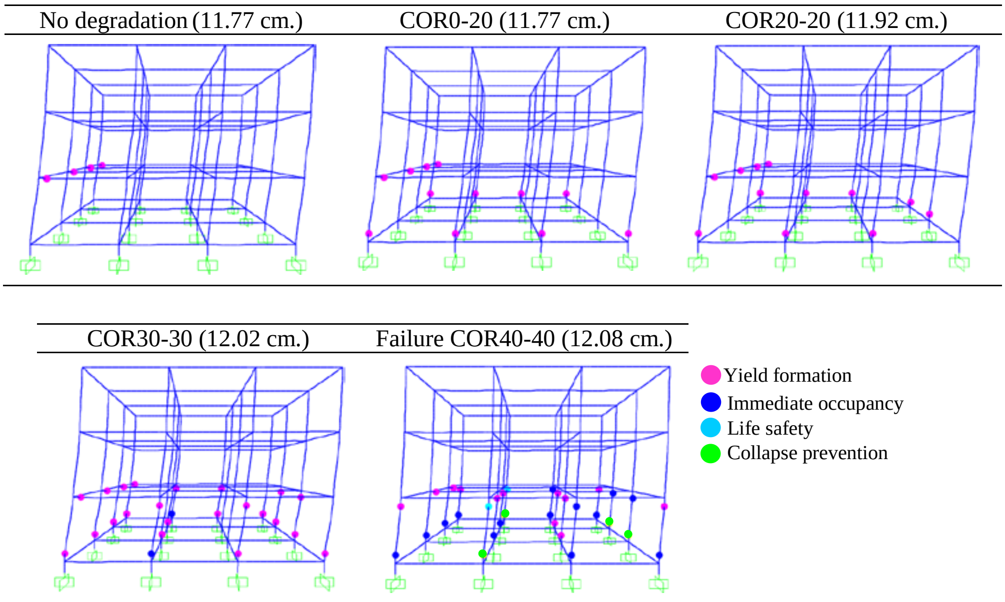

Fig.18 Formation of plastic hinges in the building under the Design Earthquake (DE).

Table 8 The displacement (Δ) and the base shear (V) at the ultimate (Δu) and the yield displacement (Δy) of the buildings.

Material Degradation	Δu [cm]	Δy [cm]	Vu [$\times 10^3$ kg]	Vy [$\times 10^3$ kg]	Vu/Vy	μ	Status at DE
No Degradation	17.50	12.72	72.73	62.91	1.15	1.38	Yield to IO
COR0-20	15.96	11.87	66.81	58.58	1.14	1.34	Yield to IO
COR20-20	15.72	12.05	65.30	58.07	1.12	1.30	Yield to IO
COR30-30	14.24	11.42	59.74	54.13	1.10	1.25	IO to LS
COR40-40	12.08	10.34	52.70	48.11	1.09	1.17	Failure (>CP)

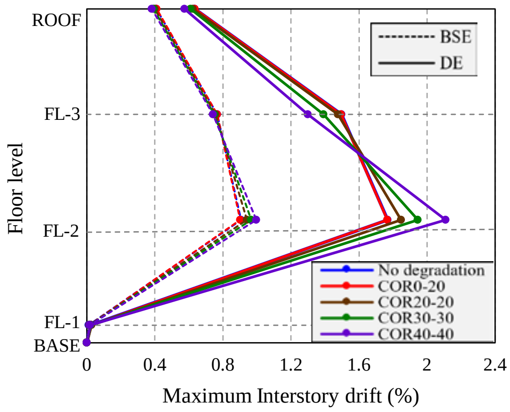

Fig.19 Maximum inter-story drift for BSE and DE.

7. CONCLUSIONS

The seismic response of a three-story residential RC building in the Northeastern region of Thailand was studied, based on different levels of material deterioration. Two levels of earthquake, including the Basic Safety Earthquake and the Design Earthquake were based on the adopted Maximum Considered Earthquake from the recent seismic hazard map. The results suggested that the buildings of all conditions were safe under the Basic Safety Earthquake. Under the Design Earthquake, the building was found to experience flexural damage in the beams, the columns and shear damage in the beam-column joints, depending on the level of material degradations. The level of material degradation affected formation of the plastic hinges and failure mechanism of the buildings. The studied RC buildings required further retrofit to achieve higher ductility.

8. ACKNOWLEDGMENTS

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