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The Effect of Compressive Strength Reduction of Column Section Expose due to Freezing-Thawing Cycles on the Seismic Performance of Bridges

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Abstract

One of the serious damages of tremendous earthquakes is the damage to bridges as the major components in an arterial road network, as relief operation is interrupted following cutting roads. Regardless of the magnitude and severity of an earthquake, other factors are also important in the strength and seismic performance of concrete bridges. Freezing-thawing cycles are among the factors, which erode the piers of concrete bridges over time. Therefore, it is necessary to evaluate the seismic vulnerability of bridges for future designs.

This research aims at discussing the effect of freeze-thaw cycles on the seismic performance of concrete bridges using fragility curves. Fragility curves express the conditional probability to reach or exceed a level of damage as a function of ground motion parameters. The curves have been developed analytically using a probabilistic method. Ground motion parameter, peak ground acceleration, structural criterion, and relative displacement of piles were considered. The non-linear time history analysis in OpenSees was used for demand determination. The curves were drawn for the slight, moderate, and extensive damage levels in two modes of before and after damage caused by thawing and freezing, i.e. the mode in which the compressive strength of column section expose reduced. With respect to the fragility curves, the strength reduction increases bridge vulnerability, especially on slight damage levels. Comparing with the cyclic curves of the most vulnerable column in two modes of before and after the damage showed that energy absorption capacity lowered with the expose compressive strength reducing.

Keywords: Concrete Bridges, Concrete Compressive Strength, Freezing-Thawing Cycles, Seismic Vulnerability, Fragility Curve, Nonlinear Time History Analysis, Damage Levels, Peak Ground Acceleration

1. Introduction

Bridges, as specific structural systems, have particularly attracted attentions of designers because their structural shape is a simple expression of their functional requirement. Therefore, the structural solutions considered for them should be confirmed as far as bridge function and elegance are concerned. Most bridges with simple structure, especially those made of RC or prestressed concrete, failed to withstand at the expected level in different earthquakes [1]. Preparing high-strength concrete, manufacturing high yield strength steel, employing high spans using modern implementation methods, easy ductility, elegance, and other concrete specifications have made concrete bridges into one of the most common bridges. On the other hand, concrete is exposed to the damages including cracking, abrasion and erosion, sulfate damage, reinforcement erosion, etc. The damage caused by continuous freezing and thawing of the water inside concrete is one of the common causes for vulnerability of concrete structures in cold climates [2]. Many studies have been conducted on the effect of freezing-thawing cycles on concrete specifications [3-8]. In 2006, Sheng *et al*. studied the strength and deformation behavior of plain concrete under 2-axial and 3-axial pressures [9, 10]. Lee and Gu estimated the lifetime of a concrete structure under thawing and freezing cycles [11].

Thawing and freezing cycles throughout the day and during years lead to continuous damage of the walls of piers of concrete bridges, which challenges the seismic performance of bridges against earthquakes. Generally, the seismic performance of the systems,

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which are expected to have nonlinear behavior, is examined using incremental dynamic analysis (IDA) [12 & 13] and nonlinear time history analysis [14 & 15]. Moreover, some methods were proposed for the dynamic analysis of bridges [16 & 17].

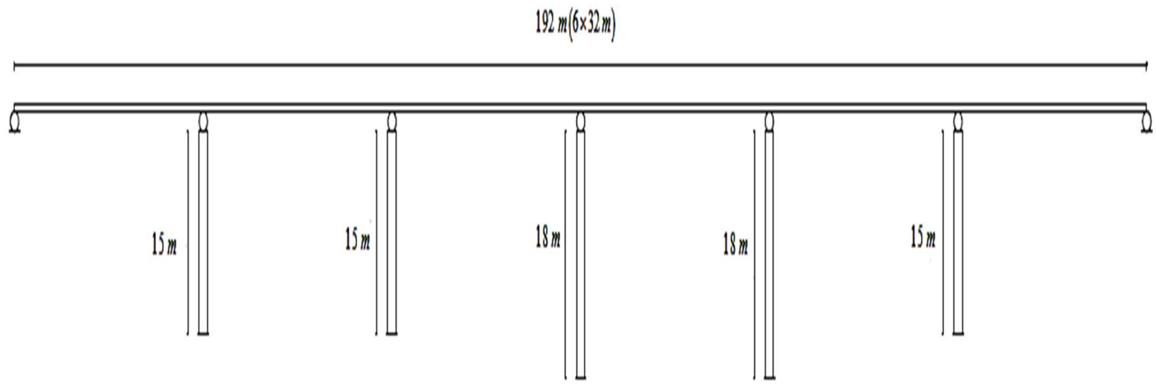
The seismic vulnerability of bridges was evaluated by analytic fragility curves. Analytical fragility curves were developed using spectral elastic analytic methods by Huang *et al*. in 2000 [18], nonlinear static analyses by Machonnes *et al*. in 2009 [19], and Liolios *et al*. in 2011 [20], nonlinear time history analyses by Huang *et al*. in 2001 [21], and Karim and Yamazaki in 2001 [22].

This study provided a 3D modeling for a concrete bridge in two modes of before and after damage caused by thawing and freezing cycles using *OpenSees* finite element [23]. The impact of thawing and freezing cycles on concrete was considered as compressive strength reduction of column section expose. After selecting the seismic records, structure responses were obtained using nonlinear time history analysis for different PGAs. To evaluate the seismic vulnerability of the bridge, fragility curves were drawn for the slight, moderate, and extensive damage levels in two modes including before and after damage caused by thawing and freezing.

2. Modeling of Bridge

This study examines a 192-meter-long six-span RC bridge. The length of each span is 32 meters. The piers of the bridge have three columns. Bridge columns are circular with the diameter of 140 cm, the expose of 7 cm, and the heights of 15 and 18 meters. Figure 1 shows the overall configuration of the bridge. As the deck structure is integrated without any expansion joint, it was modeled continuously using linear elastic elements. The columns were introduced to *OpenSees* as the string nonlinear column beam elements. The nonlinear geometric effect and P-Delta were also considered in modeling. The axial and shear stiffness of neoprenes were obtained using the relations provided by the manufacturer's catalog (*Elastomer Gumba*) and the effective stiffness of abutments was estimated by Caltrans Codes [24] and modeled by a dimensionless element.

Confined and unconfined concrete were modeled by *Concrete01 Material* behavioral model and steel was modeled using *Steel02 Material* behavioral model. Figure 1 shows the nonlinear behavior parameters of material. Twenty records of earthquakes in Iran and other countries with varied and high-durability frequency contents were selected from Berkeley University database for nonlinear time history analysis. The records consisted of far- and near-field earthquakes with the magnitudes between six and eight. The horizontal components of earthquake were applied in random and in longitudinal and latitudinal directions to the bridge structure. Specifications of the selected records are shown by Table2.



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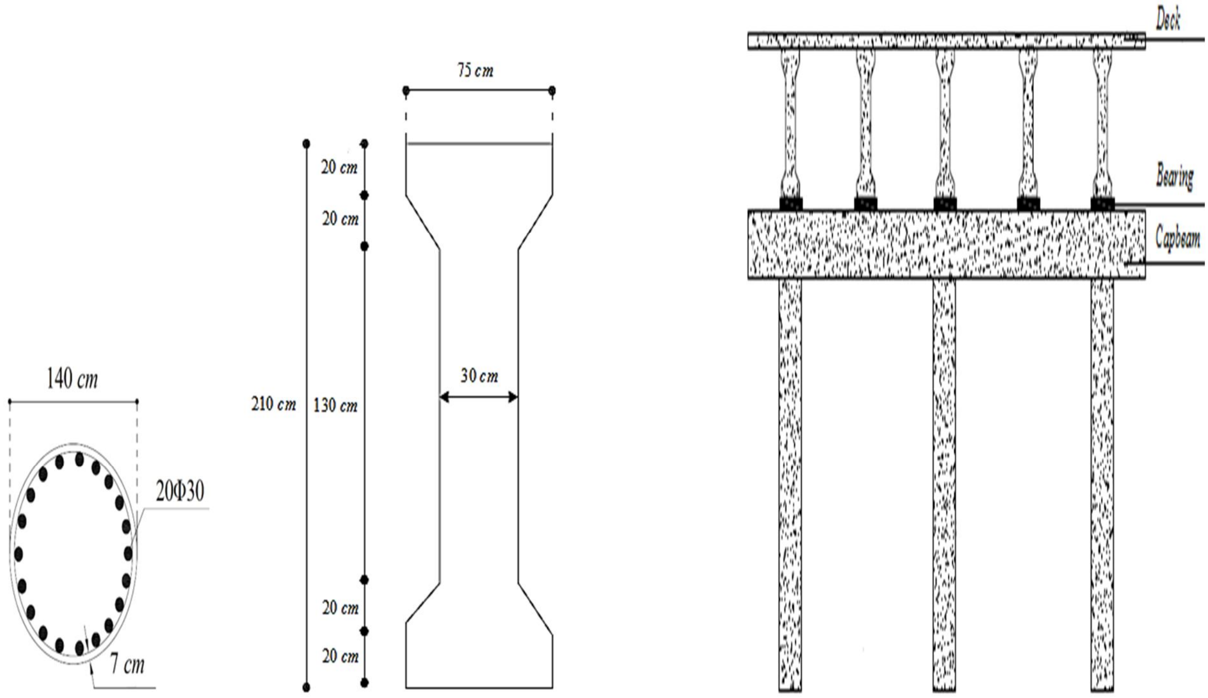


(D) (C) (B)

Figure 1. Bridge Configuration
(A) Longitudinal view of the bridge (B) Latitudinal view of the bridge, (C) Deck section, (D) Column section

Table 1. Nonlinear behavior parameters of material

Material	Nonlinear Behavior Parameters					
Steel	E_s(kg/cm^2)	F_y(kg/cm^2)	b	R0	R1	R2
	2.1×10^6	4000	0.005	18	0.925	0.15
	Fpc (kg/cm^2)	epsc0	fpcU (kg/cm^2)	epsU		
Unconfined Concrete before Damage	235	0.002	22.7797	0.004878		
Unconfined Concrete after Damage	20	0.002	2.3588	0.004878		
Confined Concrete	279.22	0.0042	228.87	0.0113		

Table 2 Specifications of the selected records

Selected Earthquake	Year of Incidence	Name of Station	Seismic Magnitude	Focal Point (Km)	$V_s 30$ (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
Whittier Narrows-01	1987	LA-116th School	5.99	23.29	301	0.3408	18.83	1.80
San Fernando	1971	Castaic - Old Ridge Route	6.61	22.63	450.28	0.2994	19.83	3.29
Northridge-01	1994	LA - Chalon Rd	6.69	20.45	740.05	0.2148	23.13	3.95
Northridge-01	1994	LA - UCLA Grounds	6.69	22.49	398.42	0.3908	22.41	5.11
Northridge-01	1994	LA -Obregon Park	6.69	37.36	349.43	0.4673	21.79	2.05
Northridge-01	1994	Castaic - Old Ridge Route	6.69	20.72	450.28	0.4898	46.51	13.57
Northridge-01	1994	Beverly Hills-12520Mulhol	6.69	18.36	545.66	0.5102	32.82	6.67
Northridge-01	1994	Santa Monica City Hall	6.69	26.45	336.2	0.5908	31.22	10.54
Cape Mendocino	1992	Eureka-Myrtle & Wes	7.01	41.97	337.46	0.1668	24.99	8.29
Cape Mendocino	1992	Rio Dell Overpass - FF	7.01	14.3	311.75	0.4244	47.95	16.96
Tabas, Iran	1978	Dayhook	7.35	13.94	471.53	0.3505	28.24	9.03
Tabas, Iran	1978	Tabas	7.35	2.05	766.77	0.8128	98.20	62.15
Taiwan Chi-Chi,	1999	CHY042	7/62	28/17	665.2	0.0823	13.71	7.23
Taiwan Chi-Chi,	1999	TCU015	7.62	49.81	426	0.1125	37.47	31.58
Taiwan Chi-Chi,	1999	NCY	7.62	13.15	599.64	0.1348	47.16	38.09
Taiwan Chi-Chi,	1999	ALS	7.62	10.80	553.43	0.1748	29.54	9.61
Taiwan Chi-Chi,	1999	TCU070	7.62	19	401.26	0.2058	56.45	51.04
Taiwan Chi-Chi,	1999	CHY029	7.62	10.96	544.74	0.2595	33.11	20.73
Taiwan Chi-Chi,	1999	TCU047	7.62	35	520.37	0.3643	38.12	36.42
Taiwan Chi-Chi,	1999	TCU095	7.62	45.18	446.63	0.5283	56.24	36.28

3. Definition of Bridge Damage Levels

This research used the method proposed by Huang *et al.* [21] to evaluate bridge vulnerability. Huang *et al.* classified five damage levels proposed in HAZUS 99 in terms of ductility-relative displacement ratio as shown by Table 3. μ_d is calculated by the following relation:

$$\mu_d = \frac{\Delta}{\Delta_{cy1}} \quad (1)$$

where Δ is the relative displacement at the top of the column, which is achieved by nonlinear time history analysis and Δ_{cy1} is the relative displacement of column when vertical reinforcement bars at the bottom of the column reach the first yield. μ_{cy1} is the ratio of ductility-first displacement of yield, μ_{cy} is the ratio of ductility-yield displacement, μ_{c2} is the ratio of ductility-displacement with 0.002 strain, and $\mu_{c\,max}$ is the ratio of ductility-maximum displacement and the moment-curvature curve specifications for column section was used for calculating them [21].

Table 3. The bridge damage levels proposed by Huang *et al*. [21]

Criteria	Damage Levels	
$\mu_{cy1} > \mu_d$	No Damage	N
$\mu_{cy} > \mu_d > \mu_{cy1}$	Slight Damage	S

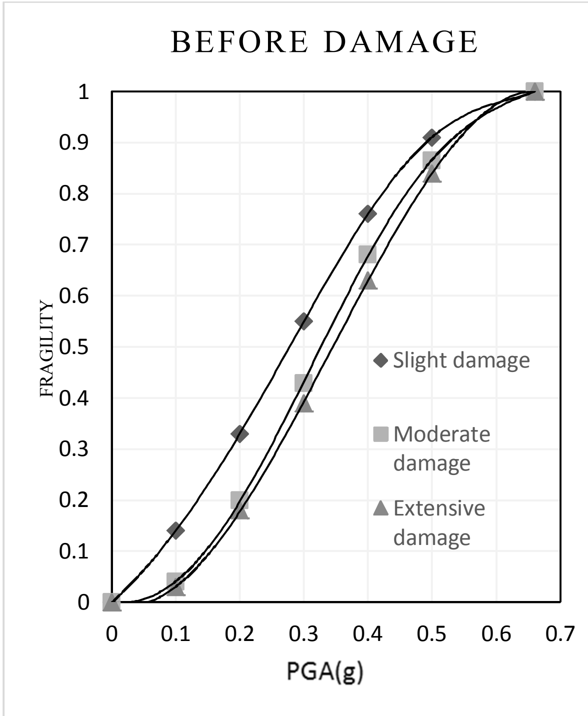
$\mu_{c\,2} > \mu_d > \mu_{cy}$	Moderate Damage	M
$\mu_{c\,max} > \mu_d > \mu_{c\,2}$	Extensive Damage	E
$\mu_d > \mu_{c\,max}$	Complete Damage	C

4. The Results of Numerical Analysis

Nonlinear time history analysis was performed for both modes of before and after damage due to freezing-thawing cycles with varied PGAs and maximum displacement of top of the columns was determined in longitudinal and latitudinal directions. (As the bearing of all columns are fixed, the relative displacement at the top of column equals the displacement of the top of column). Relation 1 was used for calculating the ratio of ductility to relative displacement in longitudinal and latitudinal directions for the modes of before and after damage due to freezing-thawing cycles. The calculated criterion was used for specifying the most vulnerable column between the longitudinal and latitudinal directions in each PGA for the modes of before and after damage due to freezing-thawing cycles.

5- Fragility Curves

After specifying the most vulnerable column in each PGA, the incidence of each damage level was calculated. For each damage level, number of incidence of the damage level and higher damage levels was counted for any PGA and the incidence of each damage level was achieved with respect to number of the input earthquakes. Fragility curves for the bridge damage levels in two modes of before and after damage in freezing-thawing cycles can be drawn using the incidence of each damage level (S, M, and E) in different PGAs. As shown by Figure 2, bridge vulnerability increased with the compressive strength of column section expose reducing due to freezing-thawing cycles. Figure 3 compares the fragility curves at any damage level of the bridge due to freezing-thawing cycles. It shows that the vulnerability increase at slight damage level exceeds the one of the moderate and extensive levels, as slight damage is related to the column scaling and it depends on concrete compressive strength, whereas moderate and extensive damages are more sensitive to bar yield stress. Therefore, with the compressive strength of expose reducing, the vulnerability at this damage level increases more than the one of other damage levels. Vulnerability increase at moderate damage level exceeds the one of the extensive damage level.



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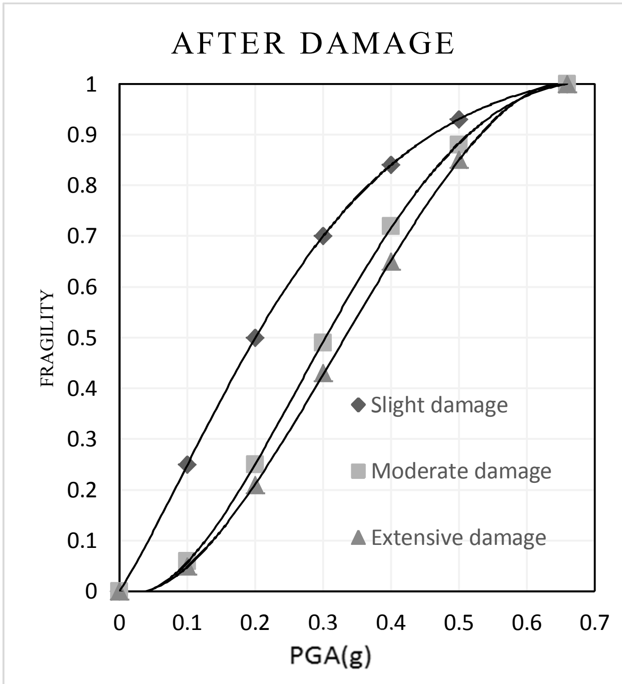

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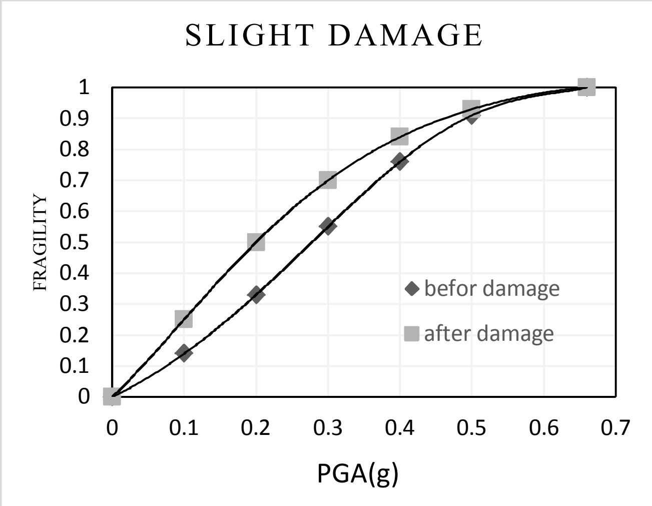
Figure 2. Fragility curves of bridge damage levels

(A) Before damage, (B) After damage due to freezing-thawing cycles



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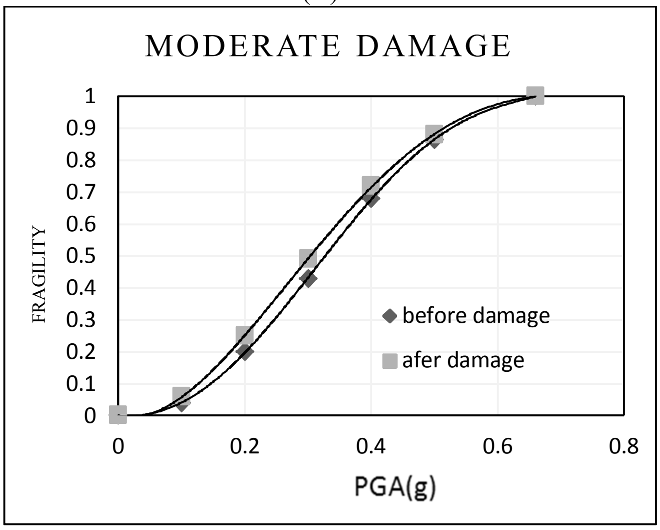

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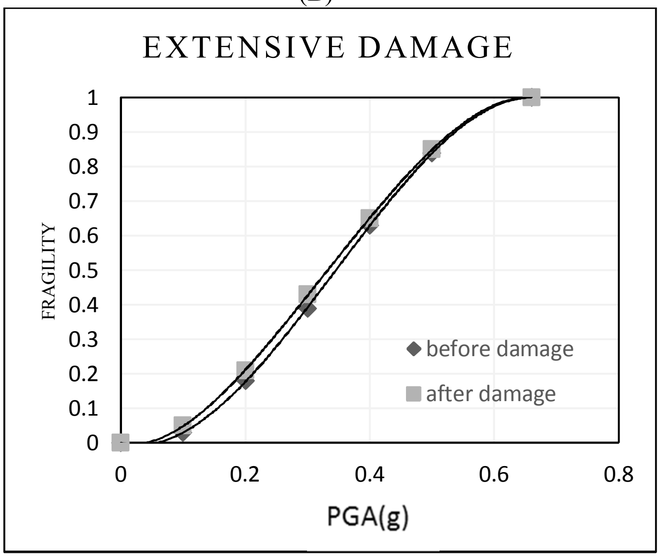


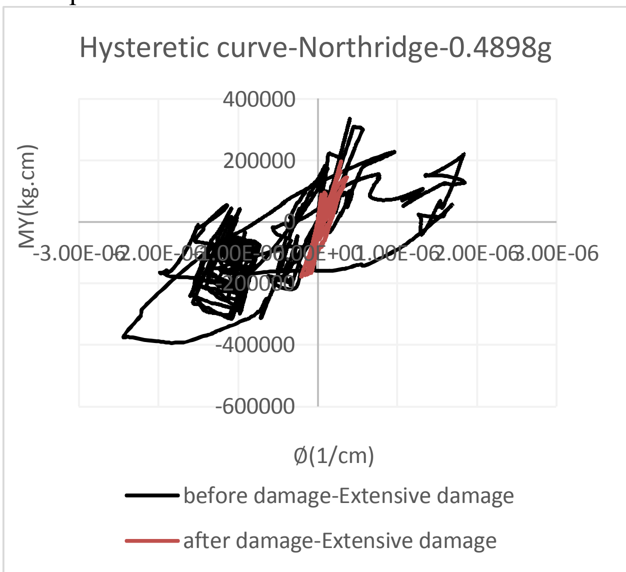
(C)

Figure 3. Comparison of fragility curves of damage levels

(A) Slight, (B) Moderate, (C) Extensive before and after damage due to freezing-thawing cycles

6. Cyclic Curves of Column

Figure 4 shows a sample of cyclic curve of the most vulnerable columns in two modes of before and after damage due to freezing-thawing cycles. It shows that with the reduction of the compressive strength of column section expose due to freezing-thawing cycles, the AUC of the cyclic curves reduce, which reduces energy absorption capacity, and this leads to inappropriate behavior of the structure under earthquake force.



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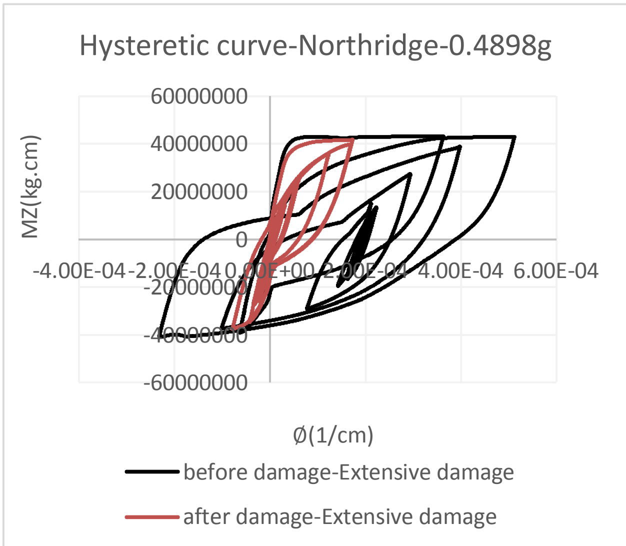

(B)

Figure 4. Comparison of the cyclic curve of the most vulnerable column under earthquake with acceleration of 0.4898g before and after damage due to freezing-thawing cycles

(A) Moment-curvature curve around Y local axis, (B) Moment-curvature aroun Z local axis

7. Conclusion

1. The results of nonlinear time history analysis showed that the study bridge is more vulnerable at latitudinal direction.
2. Bridge vulnerability increases with the compressive strength of column section expose increasing due to freezing-thawing cycles.
3. Comparison of the cyclic curves in two modes of before and after damage due to freezing-thawing cycles showed that energy absorption capacity lowers with the compressive strength of column seciton expose reducing.
4. The increase of bridge vulnerability due to reduction of concrete strength of expose at slight damage level exceeds the moderate and extensive damage levels.
5. The probability of slight damage level and over 0.5 occur in the mode before and after the damage due to freezing-thawing cycles at the peak ground accelerations of 0.28g and 0.2g, respectively. The probability of moderate damage level and over 0.5 in the modes before and after the damage occurs at the peak ground accelerations of 0.33g and 0.3g, respectively. The probability of extensive damage level and over 0.5 in the modes of before and after the damage occurs at the peak ground accelerations of 0.345g and 0.332g, respectively.

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