IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 17, Issue 6 Ser. II (Nov. – Dec. 2020), PP 01-10
www.iosrjournals.org

Effect of Reinforcement amount on the collapse pattern of RC Box Girder Bridges

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Abstract
Most of the recent studies focus on the progressive collapse of ordinary structures due to gravity and blast loads. A few focus on studying progressive collapse due to seismic actions, especially of bridge structures.The past major earthquakes have shown that it is possible to develop improved earthquake-resistant design techniques for new bridgesif the process of damage from initial failure to ultimate collapse and its effects on structural failure mechanisms could be analyzed and monitored.This paper presents a simulation and analysis of bridge progressive collapse behavior during a severe seismic action using Applied Element Method [AEM] which can take into account the separation of structural components resulted from fracture failure and falling debris contact or impact forces. A monolithic RC box girder bridge were numerically analyzed under the influence of Kobe seismic ground motion in longitudinal direction. The bridge models were tested to show the effect of reduction of the transverse reinforcement on the failure behavior of the monolithic bridge. The results showed that the collapse behavior transformed from mainly flexural failure to shear failure.
Keywords: *Progressive collapse; applied element method; box girder; Ground motion direction*

Date of Submission: 12-11-2020 Date of Acceptance: 29-11-2020

I. Introduction

Progressive collapse phenomenonis defined as the global damage or collapse behavior of a large part of the structural system that iscaused by a failure of a relatively small or localized part of the structure. StructuralProgressive collapse occurs as a result of failure of one or more structural members or components.The load is transferred in the structural system due to changes in the distribution of stiffness, the pattern of the stress behavior, and/or the structural boundary conditions (Krauthammer et al., 2002). This initial failure results in other structural elements being further overloaded and later fail.Studies on the progressive collapse of existing structures have focused primarily on high impact as in blasting or irregular loading. Not so much attention is paid to the vulnerability of structures, especially bridges, with regard to progressive collapse during earthquakes (Starossek U., 2006).

Wibowo et al., (2009) studied the seismic progressive collapse of RC bridges during earthquakes. They modeled only a continuous bridge that was previously experimented with "Guedes, 1997". The results have shown a good agreement. The separation of structural components resulting from fracture failure and impact forces from falling debris had been taken into consideration. The results have shown a significant influence on the performance of bridges during major earthquakes that were visible in its progressive collapse analysis. These also demonstrate the need to include progressive failure mechanisms in the assessment of seismic design efficiency and bridge evaluation that would not only lead to a better and more robust earthquake-resistant design for new structures but also more efficient retrofitting and reinforcement strategies for older structures.

In a similar vein, Salem et al., (2016) analyzed numerically the collapse of Tsuyagawa Bridge damaged by the Tohoku Tsunami in March 2011. The Tohoku Tsunami swept across Japan's eastern coast killing over 15,000 people and missing over 2,500. The tsunami caused more than 400,000 buildings to collapse and more than 250 coastal bridges to be washed away. The analysis showed accurately the collapse behavior of the bridge, showing that the bridge collapsed at a water velocity of 6.6 m/s caused by its piers' flexural failure. Tsuyagawa Bridge's AEM analysis has shown the ability to simulate the 2011 Tohoku Tsunami collapseeffectively, although the analytical results showed less ductility when compared to reality.

Domaneschi et al., (2020) analyzed numerically the collapse of the viaduct over the Polcevera Valley in Genoa that collapsedin August 2018. This incident left 43 deaths, and several injuries caused by a collapse of aportion of the highway connection. The results of the analysisshowed that the stay cable was the most important item whose failure caused the collapse. Furthermore, the simulation model indicated that the main girder triggered the collapse andthe large visible displacements involved in their collapse would have warned the authorities of the impending fault.

II. Applied Element Method

The Extreme Loading for Structures (ELS) program, developed by ASI-2018 is based on the AEM, whichwas initially developed by Tagel-Din and Meguro (2000a, b) at the University of Tokyo in 1998 to solve problemsrelated to two-dimensional plane stresses. It was later expanded to solve three-dimensional problems. The AEM is a novel method of modeling that adopts the discrete cracking concept in AEM.Structures are modeled as anelement assembly. The elements are not rigid and connected by normal and shear springs along their joint surfaces. These springs are responsible for normal and shear stresses transfer between adjacent elements. Each spring represents a certain volume of material stresses and deformations. (See Fig.1below). Once the connecting springs fail, each of the two adjacent elements can be completely separated. The AEM adopts fully nonlinear path-dependent material constitutive models.AEM is a stiffness-based approach in which an overall stiffness matrix is formulated and equilibrium equations for each of the stiffness, mass and damping matrices for structural deformations (displacements and rotations) are nonlinearly solved. The equilibrium equation solution is an implicit one that takes step-by-step dynamic integration (Newmark-beta time integration procedure) (Bathe 1995; Chopra 1995).If the springs connecting the elements are ruptured, two adjacent elements are separated from each other. Elements may separate, recontact, or contact other elements automaticallydepending on the structural response.See Fig. 2 below.

III. Material Models

3.1 Modeling concrete and reinforcing steel

Maekawa model is used to model concrete in compression, whereas for concrete in tension, the linear stress-strain relationship is adopted. In this stage, concrete is exposed to tension up to cracking where the stresses are set to zero afterward. Furthermore, for concrete in shear, a linear relationship between shear stresses and strain is assumed before the cracking. After cracking, a drop in the value of shear stresses to zero takes place (H. Okamura and M. Kohichi, 1991). Springs are also used to define the reinforcement between elements. Ristic model,Ristic, D., (1986) is used to model the reinforcement. Newmark-β approach is used to solve equations of dynamics. The Equilibrium equations are indeed linear for each step and are generally solved, in AEM, by using a direct or an iterative solver, Fig. 1.

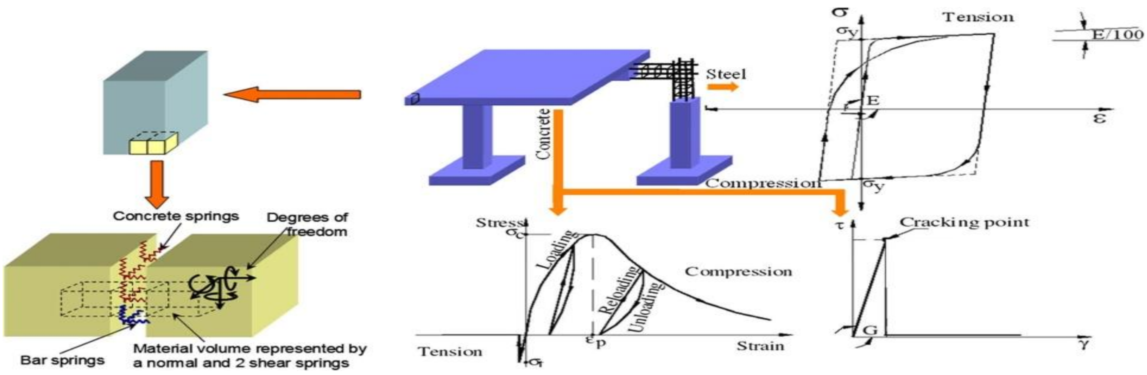


Figure1. Modeling of a structure with AEM, Salem et al. (2016).

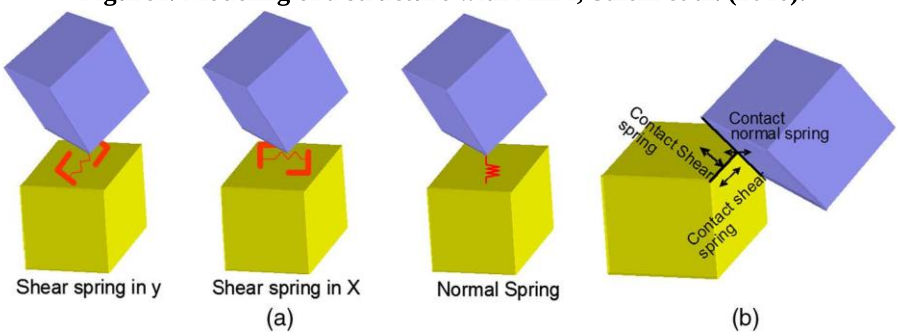


Figure 2. Different types of element contacts: (a) corner-to-face or corner-to-ground contact; (b) edge-to-edge contact, Salem et al. (2016).

3.2 Bridge bearing material

An interface material is used to model bearings. The interface material model is a pre-cracked element where the material is initially cracked and cannot bear tensile stresses. As for compression, the stress-strain relation is linear up to compression failure stress (Fig.3). The relationship between shear stress and shear strain is linear until the shear stress approaches μσn (coefficient of normal friction x normal stress). At this stress level, the shear stress remains the value (μσn) as long as there is no change in normal stresses. The compressive stress variation allows the proportional variation in shear stresses (μσn). The shear stiffness is set as a minimum, if the crack opens or during active sliding of the bearing. SeeFig. 3.(Salem et al., 2016)

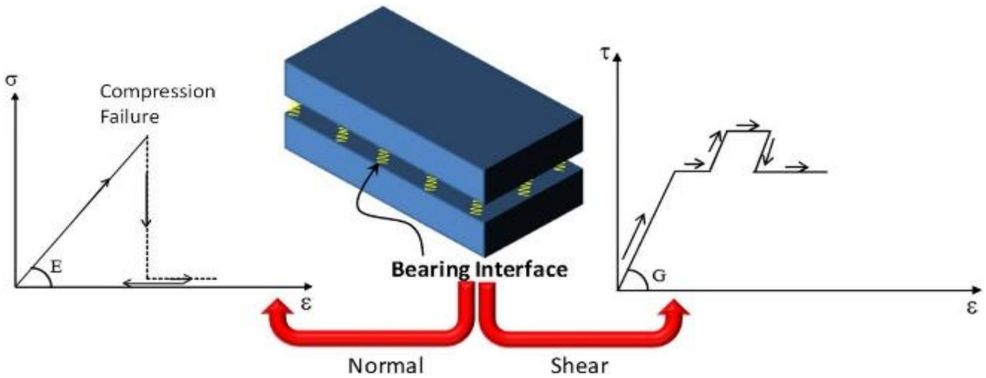


Figure 3. Modeling of a bearing interface with the AEM,Salem etal. (2016).

IV. Comparison of AEM and FEM

During progressive collapse analysis, the failure, separation, contact, and falling debris of elements must be traced. Using FEM, It is very difficult to model progressive collapse. On the other hand, using AEM, to analyze these processes is made easy and effective taking into consideration all the analysis stages until collision, Fig. 4.

	Small Displacement		Large Displacement			Collision
	Elastic	Cracking, Yield, Crushing	Buckling, Post-Bulking	Element Separation	Debris falling as rigid bodies	Progressive Collapse
	Linear	Nonlinear				
AEM	Accurate	Reliable Results				
FEM	Accurate	Reliable Results		Not Automated		Time Consuming

Figure 4. Scope of FEM and AEM.

V. Bridge Models

5.1. Bridge layout

RC box girder bridge were modeled 3 spans with25m span,Fig. 5. The bridge superstructure is monolithic box girder with columns. The columns are assumed to be fixed at its bases. The bridge box girderisrested on fiveelastomeric bearings plates at the superstructure edges. The bridge dimensions and reinforcement details were originally taken from executed multi-span box girder bridges in Egypt. The reinforced concrete damping ratio is assumed 5% during the analysis. The analyzed bridge model and the reinforcement of the box girder is shown in Fig.6, and Table 1. The purpose ofanalyzing modelsA1-L-K and A2-L-Kisto determine the effect of reduction of the transverse reinforcement under severe seismic ground motion, like Kobe, on RC box girder.

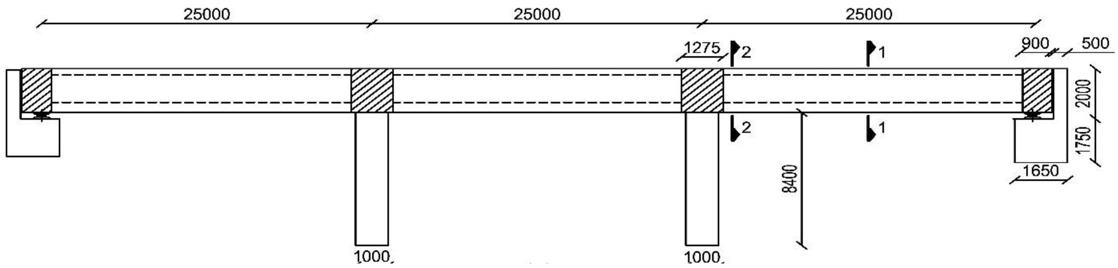


Figure5. Layout of the box girder bridges: monolithic with columns

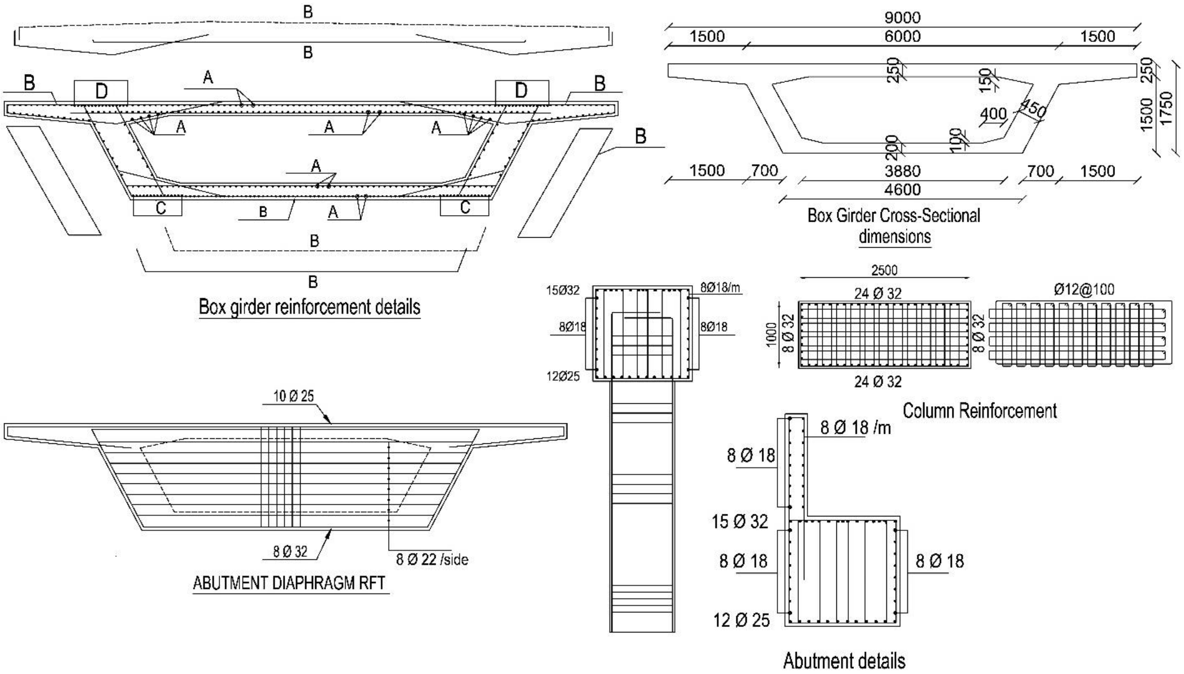


Figure 6. Dimensions of the box girder and reinforcement details of the bridge elements.

Table 1: Bridge models and box girder reinforcements (unit: mm).

Model*	Ground Motion	Bridge System	Sec.	Reinforcement of the box girder			
				A	B	C	D
A1-L-K	Kobe	Monolithic	1	Ø10/125	Ø8/125	30Ø16	10Ø16
			2				
A2-L-K			1	Ø10/125	Ø8/250	30Ø16	10Ø16
			2				

VI. Material properties

The material properties adopted inAEM analysis are presented in Table 2. A full bond between the concrete and the reinforcing steel was assumed. The used bearing was composed of a top and bottom steel plates and bearing material in between as inSalem et al., (2016). The dimensions of the steel plates used were 500x500x50 mm. The dimension of the elastomeric bearing interface was assumed 350x350x130 mm, Akogul, C. and Celik, O.,(2008). The interface between the steel plates was given bearing material properties, Salem et al., (2016). A relatively high compressive strength wasgiven to the bearing interface so it could not fail in compression and act linearly(Chen, W. F.,and Duan, L., 2014). The shear modulus of the bearing was assumed to be 2Mpa(MalekS., 2007, and Can Akogul and Oguz C., 2008).

Table 2: Properties of the bridge materials.

Parameter	Concrete	Steel Reinforcement & plates	Bearing interface	unit
Compressive Strength	4e06	3.6e07	5.51e+07	kg/m²
Tensile Strength	4e05	3.6e07	--	kg/m²
Young's Modulus	2.213e09	2.0389e+09	2.0389e+09	kgm²
Shear Modulus	984297e03	8.1556e+09	203943	kg/m²
Specific Weight	2500	7840	7840	kg/m³
Separation Strain	0.2	0.12	1	---
Friction Coefficient	0.8	0.8	0.6	---
Ultimate Strength / Tensile Stress	--	1.4444	--	---
Normal Contact Stiffness Factor	0.0001	0.0001	0.0001	---
Shear Contact Stiffness Factor	1.00e-05	1.00e-05	1.00e-05	---
Contact Spring Unloading Stiffness Factor	2	2	2	---
Post Yield Stiffness Ratio	--	0.01	--	---

VII. Ground Acceleration

Kobe, ground acceleration was used in the collapse analysis of the bridge models, as there was some bridge collapse during these earthquakes, Mitchell et al., (1995), Anderson et al., (1996), Kawashima, (2000), Wallace, et al., (2001), and Hsu and Fu, (2004). The ground motions data was obtained from the Pacific Earthquake Engineering Research (PEER), Strong Motion Database(PEER, 2019). A summary of the earthquake ground motion used in this research is presented in Table 4 and is shown in Fig. 7. The time used in the seismic analysis was reduced to the time that contains the largest cycles of seismic accelerations to reduce the ELS analysis time, as the time that would not contain significant values of acceleration couldbe omitted. The used time step during the analysis was 0.004. Earthquake analysis usually requires ΔT of 0.001-0.01 sec. when a collision is expected to occur. The smaller the time step the higher the accuracy and the convergence of results maintained.

Table 4: Seismic ground motions.

Earthquake	Year of Occurrence	Record Station	PGA in X-Dir.	Moment Magnitude	Original Duration	Reduced Duration
Kobe	**Jan 1995**	**KJMA**	**0.834g**	**6.9**	**90**	**20**

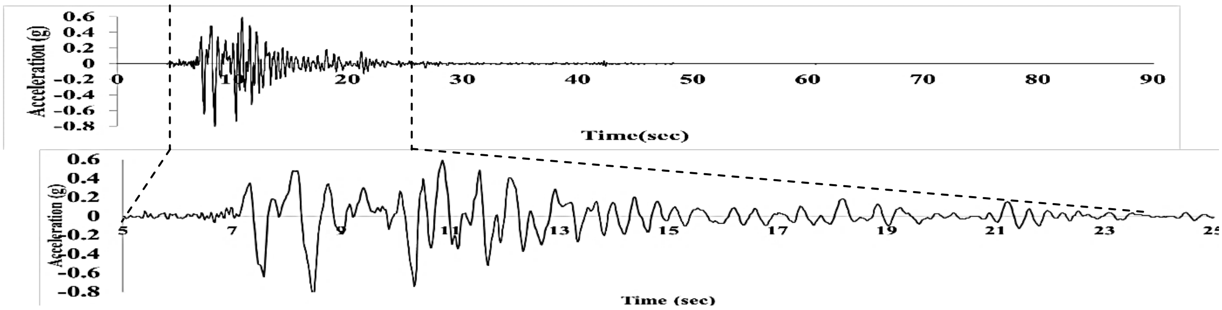
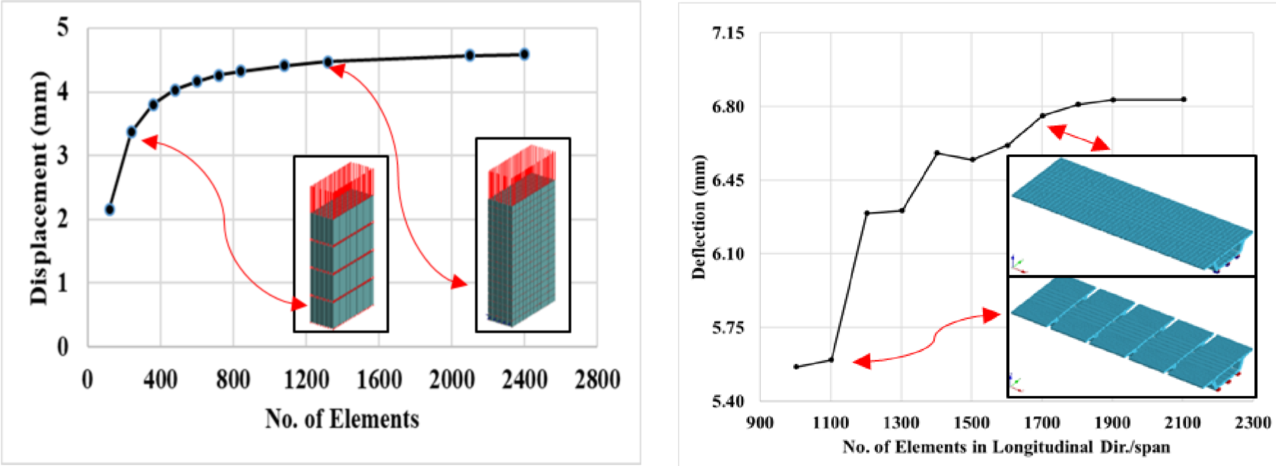


Figure 7 Original and reduced 1995, Kobe earthquake ground motion.

VIII. Mesh Sensitivity Analysis

A mesh sensitivity analysis was carried out to obtain a suitable mesh size that would be used in all the analysis cases for columns and bridge superstructure. Horizontal and vertical concentrated loads were used for the column and the box girder respectively. Fig. 8 shows the relationship between the mesh elements and the displacement of the column and the deflection of the box girder. 22 elements per column`s height and 5x12 elements per columns` cross-section were used. The maximum dimensions for the columns` elements were 200x200 mm per elementcross-section andwas 38 cm per elementheight. Each surface area of the box girder (i.e., the deck, soffit, and webs) wasdivided into 5x1 elements with 50 elements per 25 m length (span) in the box girders` longitudinal direction. This mesh size wasfound to give accurate results. An analysis using a finer mesh has been carried out without any noticeable difference in the displacement and deformation. The total number of elements used was 10,000,.The AEM mesh used was accurate enough during the elastic region and in the small deformation range of the inelastic region, Tagel-Din and Meguro (2000a, b).



(a) Column **(b) Box Girder**

Figure 8. Mesh sensitivity of the column, and the box girder.

IX. Analysis Results

The analysis was carried out on two stages; the first was static to take into account the gravity loads and original deformations of the bridge,whereas the second was a dynamic analysis.

9.1. Reinforcementreduction effect

Fig. 9 shows the displacement time history for the right column ofA1-L-K, A2-L-K, models. The two modelsshowed a relatively identical behavioruntil the 5th second, as there was no failure in both bridge models. at the 5th second, model A2-L-K showed a shear failure of the right bay of the bridge box girder,which in turn produced a higher displacement, 150 mm, than model A1-L-K till the end of the analysis at the 20th second, the middle pay of model A1-L-K collapsed and the column exhibited large displacement, as the middle bay dragged the column down to the earth, which nearly equal-300 mm.

Fig. 20 shows the straining actions. The straining actions of models A1-L-K, A2-L-Kwere nearlyidentical and the difference in curves was produced from the early collapse of the box girder of model A2-L-K. after the collapse of the right bay of the box girder of the two models, A1-L-K, A2-L-K, the axial force of the right column was reduced to 50% as the column is still loaded from the left span of the box girder. At the 18th second, model A1-L-K showed an abnormal straining actions, as the left box girder collapsed and the column became released.

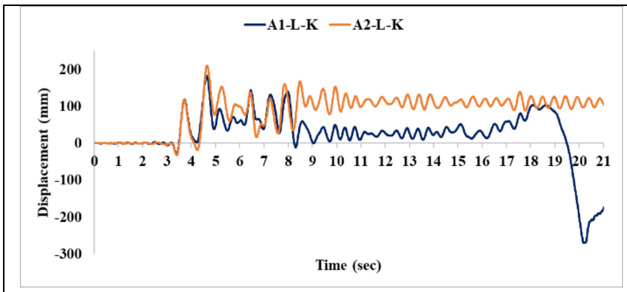


Figure 9. Displacements-time history of the right column of models “A1-M-L, and A2-M-L,”

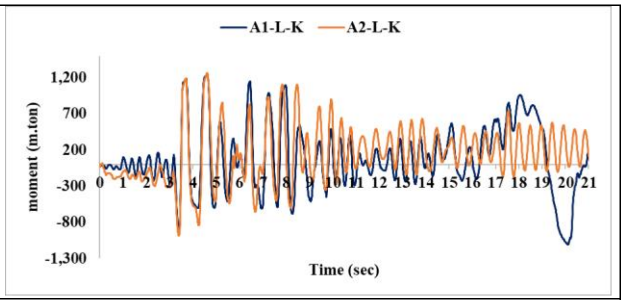


Figure 10.b. Moment-time history of the right column of models “A1-M-L, and A2-M-L,”

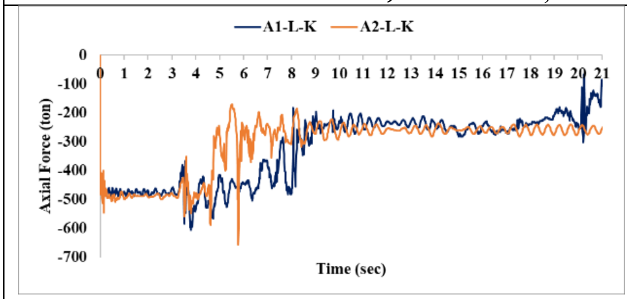


Figure.10.a. Axial force-time history of the right column of models “A1-M-L, and A2-M-L,”

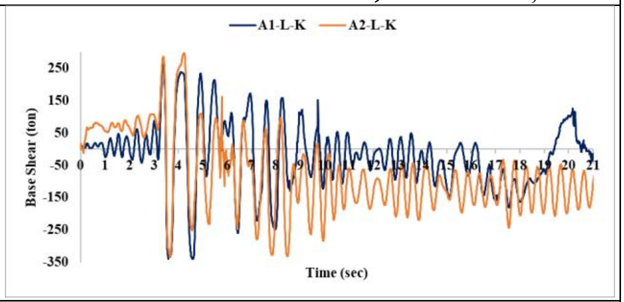


Figure 10.c. Base shear-time history of the right column of models “A1-M-L, and A2-M-L,”

9.3. Collapse analysis of the different bridge models during Kobe ground motion

A comparison between models A1-L-K, A2-L-Kare presented in Figs. 11, and 12respectively.

The less the bridge superstructure reinforcement, the more cracks or collapse observed in the box girder. By reducing the amount of transvers reinforcement in the box girder, the collapse pattern transformed from flexural failure, in A1-L-K ,to shear failure, in model A2-L-K, At the end of the analysis time. it is noted that model A1-L-K that failed in flexure had takennearly 17 sec to collapse. However, by reducing the transverse reinforcement in model A2-L-K had taken around 7 seconds.

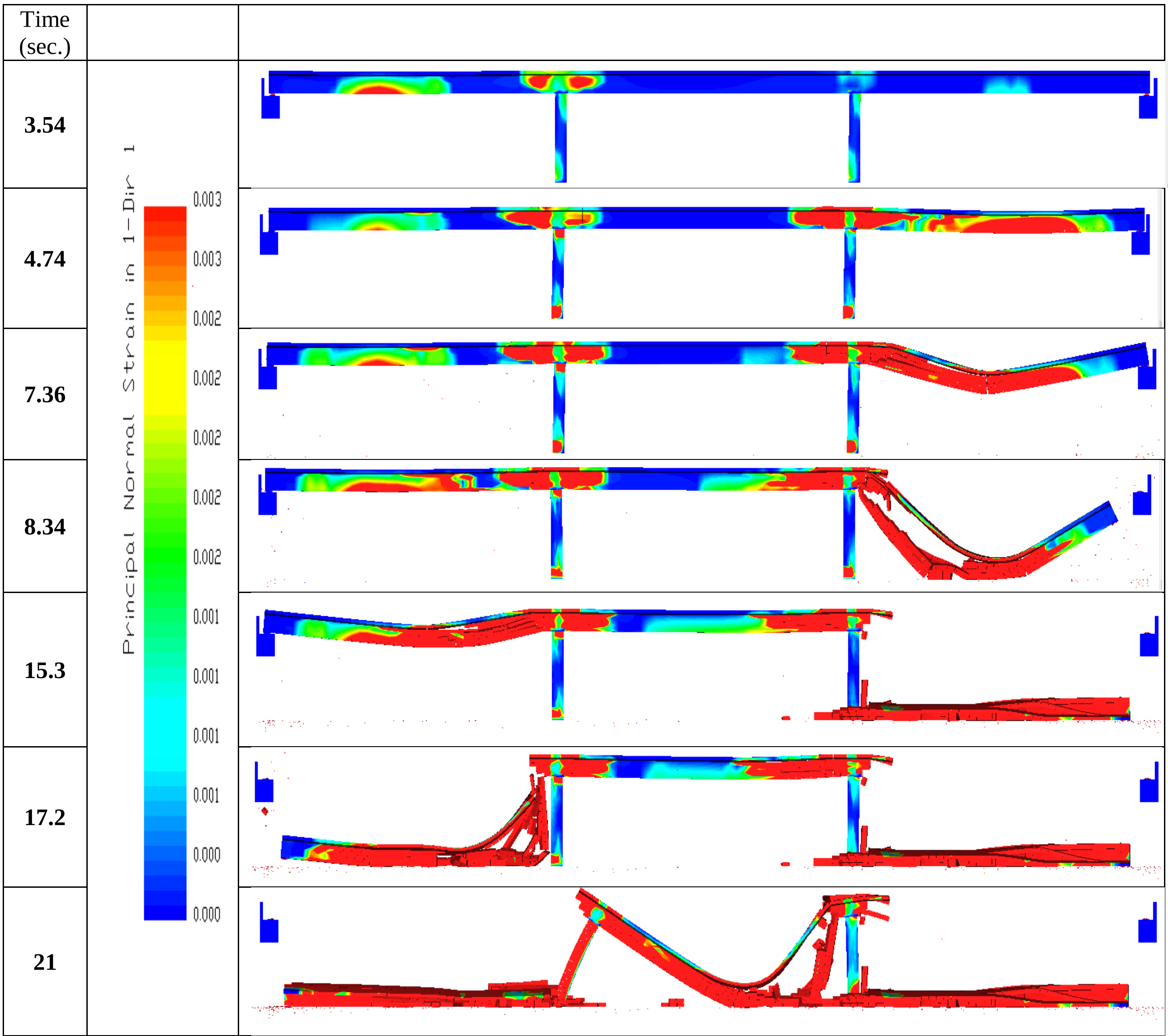


Figure 11.a. 2D-view of the principal normal strain during the time history “Model A1-L-K”.

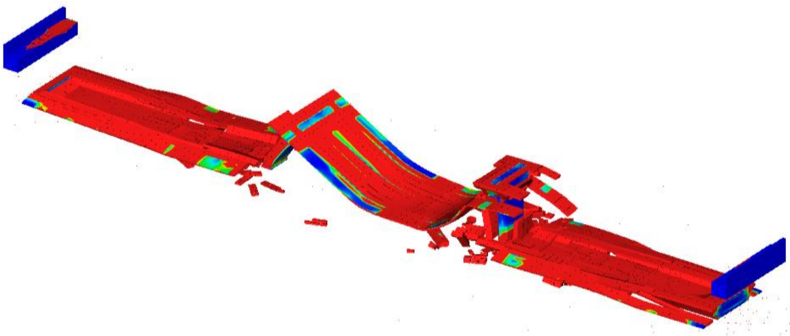


Figure 11.b. 3D-view of the principal normal strain during the time history “Model A1-L-K”.

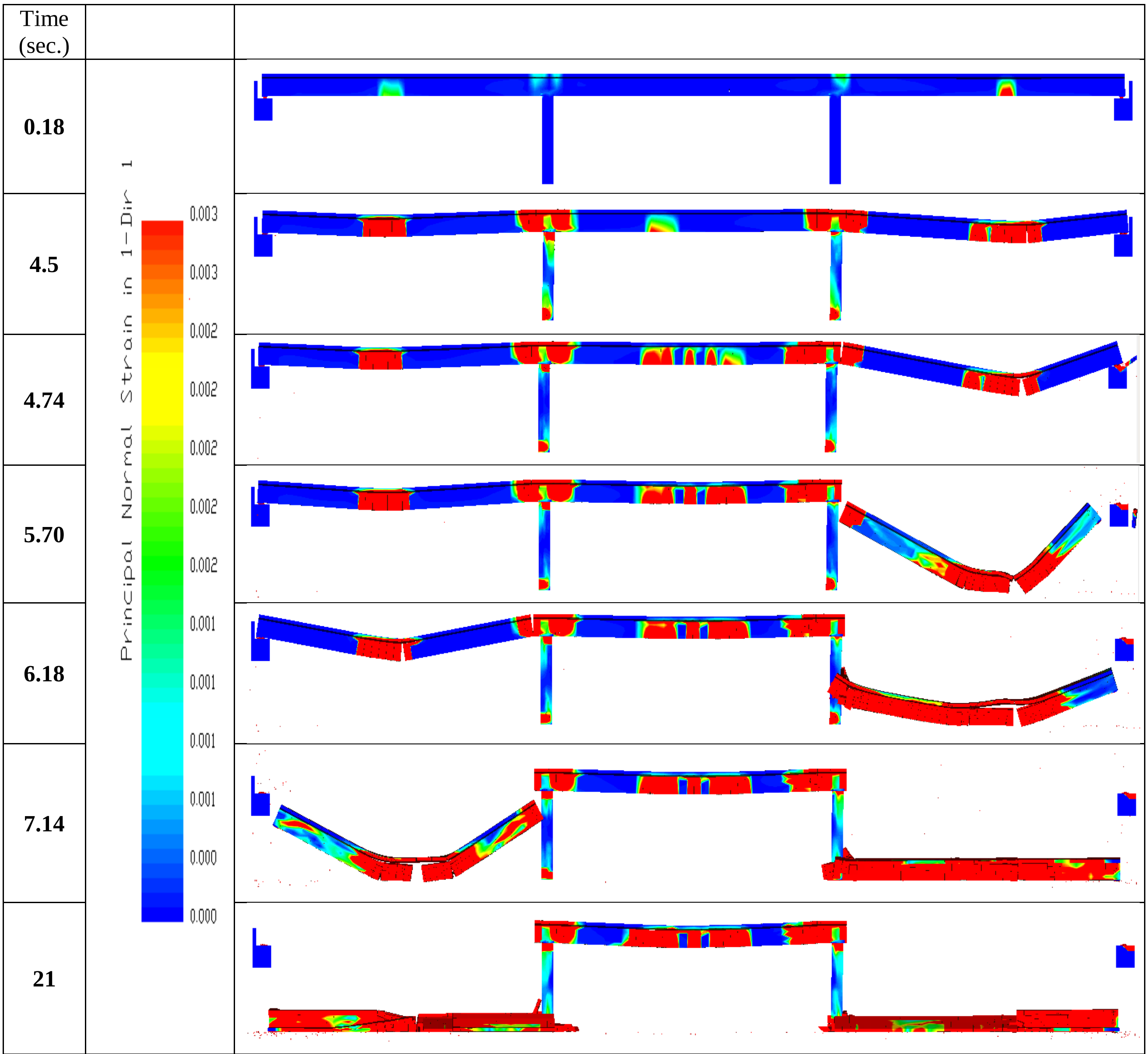


Figure 12.a. 2D-view of the principal normal strain during the time history “Model A2-L-K”.

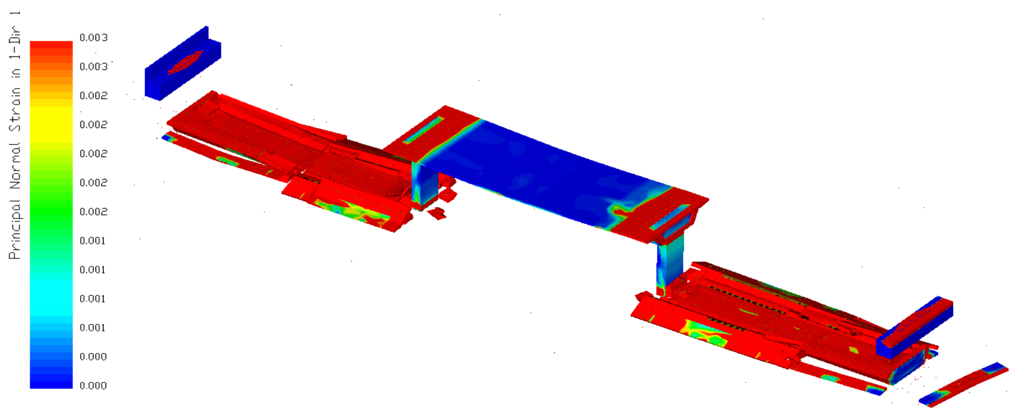


Figure 12.b. 3D-view of the principal normal strain during the time history “Model A2-L-K”.

X. Conclusion

In the current study, the seismic progressive collapse behavior and analysis of reinforced concrete bridges were analyzed. Various bridge configurations: monolithic with columns, continuous on bearings, simple on bearings bridge models were analyzed. The bridge models and selected earthquake excitations used in the study were discussed. A summary of the findings is presented herein.

- ELS program can be a means to predict the behavior of ordinary and special structures against abnormal events during the design, construction, and service loads.
- Bridges can be analyzed using the actual amount of reinforcement, obtaining the collapse pattern, and analyzing the necessary strengthening to prevent the possibility of collapse.
- By reducing the amount of transvers reinforcement in the box girder, the collapse pattern transformed from flexural failure, in A1-L-K ,to shear failure, in model A2-L-K.
- Changing the amount of reinforcement in bridges can change the collapse pattern and, which can be used produce a collapse that does not cause great losseshuman lives.

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M. M. Husain, et. al. "Effect of Reinforcement amount on the collapse pattern of RC Box Girder Bridges." *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)*, 17(6), 2020, pp. 01-10.