MODELLING OF SELF ACTUATING MR DAMPERS FOR BRIDGES

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Abstract:* Earthquake damage to a bridge can have severe consequences. Clearly, the collapse of a bridge places people on or below the bridge at risk, bridges are often provided as a link in a transportation system. so it is important to prevent the bridge from the collapse. In this paper, 91/5 over-crossing, located in Orange County of Southern California is taken as a benchmark bridge with the provided literatures and it is modeled and time history analysis is done. In order to prevent the bridge from collapse, displacement is to be controlled and reduced. Magnotorheological (MR) dampers have been demonstrated to be more effective in reducing the structural response due to earthquakes using only a small amount of external power. In this paper MR dampers are used to control the structural responses from bridge and in-order to make the semi-active damper into passive damper.

Key words: **Seismically excited highway bridge; CSI bridge; Earthquake response control; Smart protective systems.**

I. INTRODUCTION

Roads are the lifelines of modern transport, and bridges are an integral part thereof. They are susceptible to failure if their structural deficiencies are unidentified. A large number of bridges constructed around the world were designed during the period, when bridge codes had no seismic design provisions, or when these provisions were insufficient according to the current standards.

Buildings have a high degree of redundancy generally inherent in their structural system, which enables alternative load paths to be mobilized, whereas bridges have little or no inherent redundancy. As earthquakes can easily identify the structural weaknesses and concentrating damages at the weakened locations, the failure of one structural component or connection between the elements in the bridges is more likely to cause the collapse of the entire bridge structure, unlike in buildings. The failures of bridges during the recent earthquakes have created an awareness, to evaluate the structural vulnerability of the bridges which were built before 2001, under seismic ground motions, to develop the required retrofit measures.

In order to reduce the vulnerability of building and bridge structures to severe earthquakes, 'smart' base isolated structures, where the performance of the base isolation system is improved by adding semi-active variable stiffness and damping devices, have been proposed and studied by various researchers.

It is extremely critical that these bridges remain operational following severe earthquakes. The condition of highway bridges in transportation infrastructure is a critical factor influencing national productivity and ability to compete in the international economy. Kawashima K, Unjoh S[3] proposed variable damper to control bridge response he demonstrated that the peak deck displacement and acceleration are reduced to 26% and 44% and Symans MD, Kelly SW[4] he examined through an analytical and computational study of the seismic response of a bridge structure containing a hybrid isolation system consisting of elastomeric bearings and semi-active dampers.

Murat Dicleli[5] discussed the merits of a hybrid seismic isolation system used for the seismic design of a major bridge The concept of seismic base isolation has been adopted into practice with the development of natural rubber bearings Sanjay.S.Sahasrabudhe,Satish Nagarajaiah[6] has explained the Sliding base-isolation system in bridges to reduce pier drifts.Tsopelas P, Okamoto S, Constantinou and Makris N, Zhang J[7] has explained that Semi-active controllable non-linear dampers which can vary damping appropriately in real time can reduce bearing displacements and forces further than the passive dampers.

Anil Agrawal,Ping Tan,Satish Nagarajaiah[1] explained that it is important to investigate the comparative effectiveness of various protective systems in reducing response quantities of highway bridges and they have developed the 3D finite-element model of the highway bridge, design of sample controllers, prescribed ground motions and a set of evaluation criteria. Ping Tan and Anil K. Agrawal[2] developed a reduced-order model of the system is developed for the design of active and semi-active controllers.

A. Magnetorheological damper

In recent years, MR damper is identified as a potential device for semi-active control for building frames because of its mechanical simplicity, low power requirement, high dynamic range, large force capacity, and robustness. Being an energy dissipation device that cannot

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add mechanical energy to the structural system, an MR damper is also very stable and fail-safe.

A magnetorheological damper or magnetorheological shock absorber is a damper filled with magnetorheological fluid, which is controlled by a magnetic field, usually using an electromagnet. This allows the damping characteristics of the shock absorber to be continuously controlled by varying the power of the electromagnet.

The Magneto-Rheological (MR) fluid damper is a promising device for civil structures due to its mechanical simplicity, inherent stability, high dynamic range, large temperature operating range, robust performance, and low power requirements. The MR damper is intrinsically nonlinear and rate-dependent.

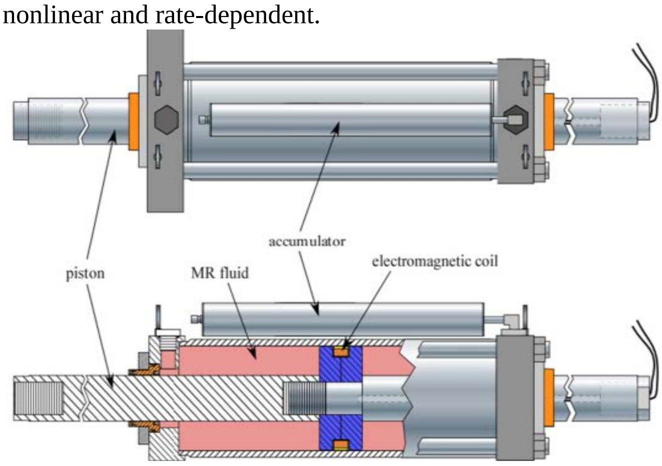

Fig 1 Magnetorheological damper

II. BENCHMARK HIGHWAY BRIDGE

The highway bridge used for this benchmark study is the newly constructed 91/5 over-crossing, located in Orange County of Southern California.

It is a continuous two-span, cast-in-place prestressed concrete box-girder bridge. The bridge has two spans, each of 58.5m (192 ft) long, spanning a four-lane highway and has two abutments skewed at 33degree. The width of the deck along east span is 12.95m (42.5 ft) and it is 15m (49.2 ft) along the west direction. The cross section of the deck consists of three cells. The deck is supported by a 31.4m (103 ft) long and 6.9m (22.5 ft) high prestressed outrigger, which rests on two pile groups, each consisting of 49 driven concrete friction piles. The columns are approximately 6.9m (22.5ft) high.

The pile groups at both end abutments consist of vertical and battered piles. The effects of soil–structure interaction at the end abutments/approach embankments are considered. The ground motions are considered to be applied to the bridge simultaneously in two directions.

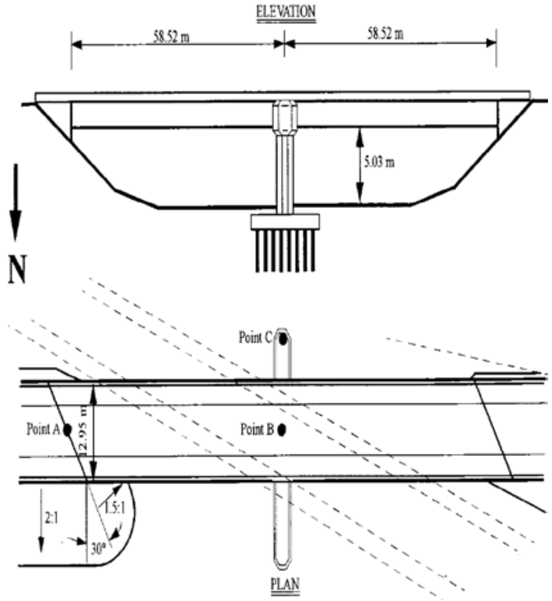

Fig 3 Elevation and plan views of 91/5 over-crossing.

III BENCHMARK EARTHQUAKE GROUNDMOTION DATA

A. Imperial valley:

The earthquake was 6.4 on the moment magnitude scale, with a maximum perceived intensity of IX (Violent) on the Mercalli intensity scale. However, most of the intensity measurements were consistent with an overall maximum intensity of VII (Very strong), and only the damage to a single structure, the Imperial County Services building in El Centro, was judged to be of intensity IX.

B. Elcentro:

The 1940 El Centro earthquake (or 1940 Imperial Valley earthquake) occurred at 21:35 Pacific Standard Time on May 18 (05:35UTC on May 19) in the Imperial Valley in southeastern Southern California near the international border of the United States and Mexico. It had a moment magnitude of 6.9 and a maximum perceived intensity of X (Extreme) on the Mercalli intensity scale.

The Salton Trough is part of the complex plate boundary between the Pacific Plate and the North American Plate where it undergoes a transition from the continental transform of the San Andreas Fault system to the series of short spreading centers of the East Pacific Rise linked by oceanic transforms in the Gulf of California. The two main right lateral strike-slip fault strands that extend across the southern part of the trough are the Elsinore Fault Zone/Laguna Salada Fault to the western side of the trough and the Imperial Fault to the east. The Imperial Fault is linked to the San Andreas Fault through the Brawley Seismic Zone, which is a spreading center beneath the southern end of the Salton Sea.

C. Northridge:

The 1994 Northridge earthquake occurred on January 17, at 4:30:55 a.m. PST and had its epicenter in Reseda, a neighborhood in the north-central San Fernando Valley region of Los Angeles, California. It had a duration of approximately 10–20 seconds. The blind thrust earthquake had a moment magnitude (M_w) of 6.7, which produced ground acceleration that was the highest ever instrumentally recorded in an urban area in North America, measuring 1.8*g* (16.7 m/s^2) with strong ground motion felt as far away as Las Vegas, Nevada, about 220 miles (360 km) from the epicenter. The peak ground velocity at the Rinaldi Receiving Station was 183 cm/s (4.09 mph or 6.59 km/h), the fastest peak ground velocity ever recorded. In addition, two 6.0 M_w aftershocks occurred, the first about one minute after the initial event and the second approximately 11 hours later, the strongest of several thousand aftershocks in all.

IV. MODELING AND ANALYZING OF BRIDGE

A. Modelling and Analyzing of bridge:

With the provided detailing of the bridge modelling and analyzing is done using the software csi bridge.

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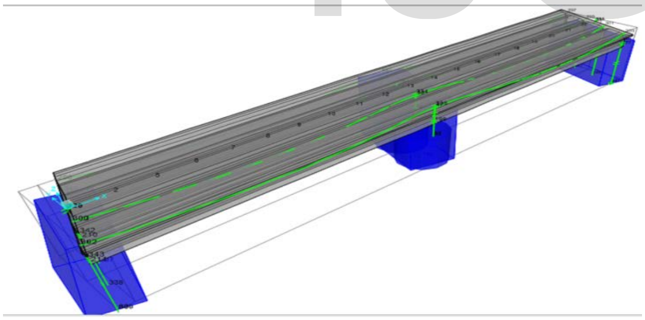

Fig 3 View of 91/5 highway over-crossing in CSi bridge

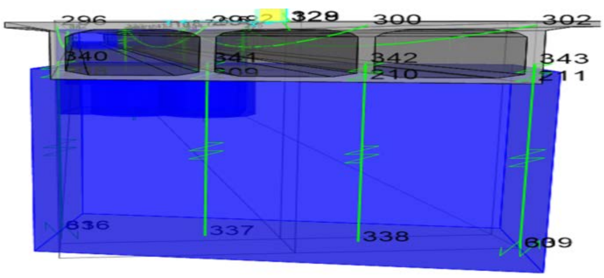

Fig 4 Internal girders in bridge

B. Time history analysis:

As per the conventional earthquake-resistant design philosophy, the structures are designed for forces, which are much less than the expected design earthquake forces. Hence, when a structure is struck with severe earthquake ground motion, it undergoes inelastic deformations. Even though the structure may not collapse but the damages can be beyond repairs. In reinforced cement concrete (RCC) structures, a structural system can be made ductile, by providing reinforcing steel according to the IS:13920-1993 code.

A sufficiently ductile structural system undergoes large deformations in the inelastic region. In order to understand the complete behaviour of structures, time history analysis of different Single Degree of Freedom (SDOF) and Multi Degree of Freedom (MDOF) structures having non-linear characteristics is required to be performed. The results of time history analysis, i.e. non-linear analysis of these structures will help in understanding their true behavior. From the results, it can be predicted, whether the structure will not collapse / partially collapse or totally collapse.

Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non linear.

Time- history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by:

$$K\,u(t) + C\,\dot{u}(t) + M\,\ddot{u}(t) = r(t)$$

where K is the stiffness matrix; C is the damping matrix; M is the diagonal mass matrix; u, u&, and u&& are the displacements, velocities, and accelerations of the structure; and r is the applied load. If the load includes ground acceleration, the displacements, velocities, and accelerations are relative to this ground motion.

Any number of time- history Load Cases can be defined. Each time-history case can differ in the load applied and in the type of analysis to be performed.

V. RESULTS AND DISCUSSION

Modelling of bridge is done using CSi bridge software and non linear time history analysis is done.

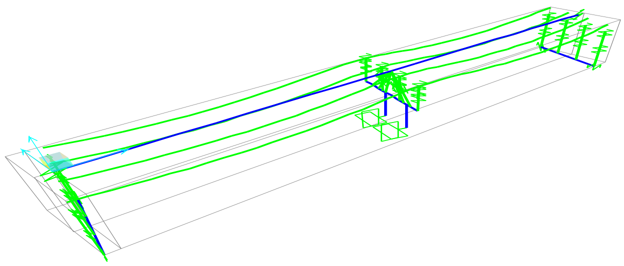

Figure 6.1: Finite element model

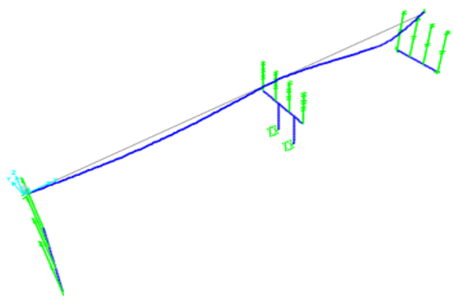

Figure 6.2 deformed shape

A.Mode shapes and frequencies:

Each nodal mass of the deck and the bent was assigned six dynamic degrees of freedom (DOF). The deck-ends and abutments, which are assumed to be infinitely rigid in plane, are modeled using three master DOF (two translational and one torsional DOF).So the six degrees of freedom are considered and six mode shapes are taken for the reference.

First mode shape attained at a time period of 0.1306sec with the frequency of 7.625cycle/sec and the second mode shape happened due to vertical at 0.1262sec and the frequency of the second mode shape is 7.91799cycle/sec. But the third mode shape occurs to be prior to the time period of second mode shape i.e at a time period of 0.0741sec with the frequency of 13.4899cycle/sec following fourth,fifth and sixth mode shape occur consecutively with the time period of 0.0556sec,0.00515sec and 0.0441sec with frequency of 17.982cycle/sec,19.401cycle/sec and 22.663cycle/sec.

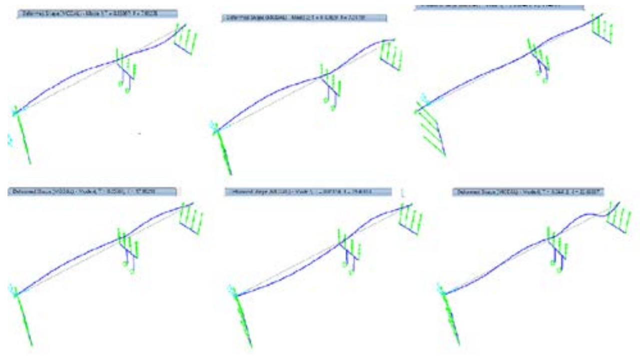

Fig 6.3 Mode shapes

Mode	Mode no	Period(sec)	Frequency(cycle/sec)
Mode	1	0.13	7.65
Mode	2	0.12	7.91
Mode	3	0.07	13.49
Mode	4	0.055	17.98
Mode	5	0.051	19.40
Mode	6	0.044	22.66

Table 17 Time period and frequencies

B. Bridges with damper:

The bridge deck is isolated using bearings at each abutment, and the total eight fluid dampers are installed between the end abutments and deck (four dampers at each end) and four dampers between bent and deck to reduce seismic responses.

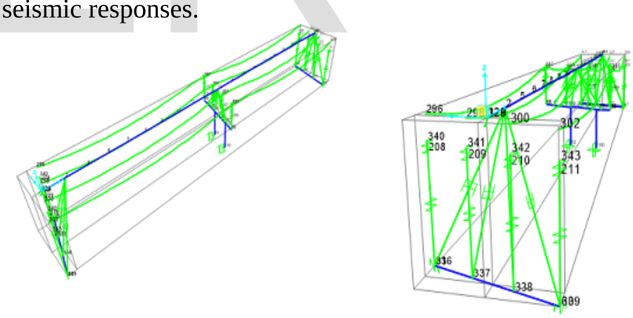

Fig 6.4 Finite element model of bridge with damper

C. Result comparisons:

1. Displacements:

Earthquake	Without damper	With damper
Elcentro	**0.00166**	**0.00148**
Imperial	**0.0001225**	**0.0001081**
Northridge	**0.0002454**	**0.0001801**

Table 18 Comparison of displacements with damper and without damper

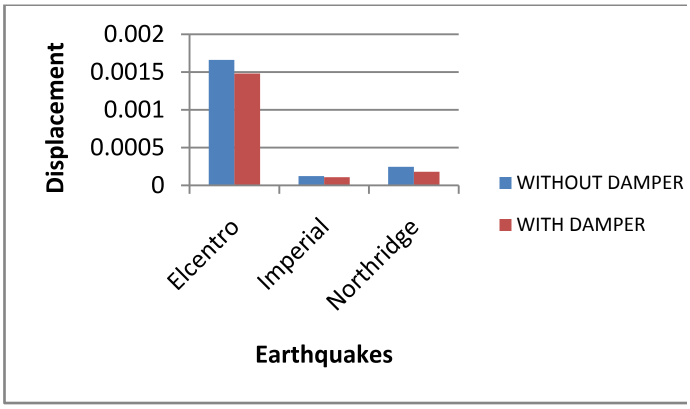

Fig 6.20 Displacement comparison with and without damper

2. Base shear:

Earthquake	Without damper	With damper
Elcentro	21811.92	18753.114
Imperial	1512.43	1362.86
Northridge	4779.85	4678.85

Table 19 Comparison of base shear with damper and without damper

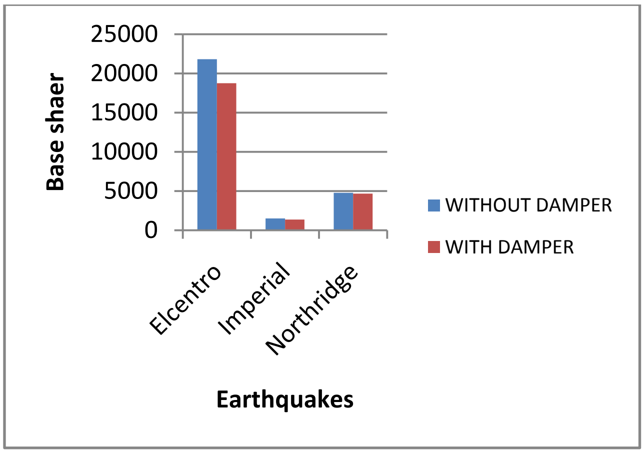

Fig 6.21 Comparison of base shear with and without dampers

VI.CONCLUSION

In this research, a newly constructed benchmark bridge has been modeled using CSi bridge software and non-linear time history analysis is done for earthquake ground motion data and its analyzed the benchmark bridge with and without magneto rheological dampers and we have found that the displacement has been reduced of 26.6 percentage for Northridge earthquake ground motion data.

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