Heavy Movable Structures, Inc.

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"Seismic Performance of Movable Bridges"

by

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SEISMIC PERFORMANCE OF MOVABLE BRIDGES

by

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With increased attention being given to the seismic performance of bridges, one needs to include the seismic performance of movable bridges in this performance evaluation.

Current seismic design criteria for highway bridges is reflected in the AASHTO Standard Specification for Highway Bridges, Section IA and the AASHTO Load and Resistance Factor Standard Specifications for Highway Bridges. Both were based on guidelines prepared by the Applied Technology Council in the early 1980's and based on an earthquake with a 500 year return period (one with a 90% probability of not being exceeded in 50 years). As stated in AASHTO, these provisions apply to conventional highway bridges and do not apply to movable bridges. The seismic design of railroad bridges is covered in the AREA Manual, Chapter 9, where it is also indicated that the design provisions do not apply to movable bridges. Nevertheless one can also conclude that the base acceleration maps included in the relevant specifications do apply.

The only specifications that specifically apply are the AASHTO Standard Specifications for Movable Highway Bridges, 1988, which state that seismic loads for movable bridges shall be as specified in the current AASHTO Standard Specifications. They also state that movable bridges that are in one position over 90% of the time, open or closed, can be designed for one half the seismic load in the other position. No explanation is offered although the intent is clear that the reduced force is related to the lower probability that a bridge will experience a seismic event when in the other position.

And all of these provisions apply to the design of new bridges. For existing bridges the FHWA Publication, Seismic Retrofitting Manual for Highway Bridges, 1995, gives guidance for existing highway bridges but again does not include movable and railroad bridges.

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And even these specifications and guidelines have their shortcomings with the most significant being the hazard maps utilized by AASHTO. The maps in AASHTO were based on seismic hazard level studies over 10 years ago and were limited to 500 year return periods. The AREA guidelines, which were published more recently, do include several hazard levels, including 100, 475 and 2400 year return periods. However, more recent information that is now available from the United States Geological Survey (USGS) has some significant differences from that previously used by AASHTO and AREA. If you are curious you can go to the USGS web site and download site specific information based on a Zip Code. The information is available for several hazard levels and several spectral periods.

Figure 1 illustrates this point for a 500 year hazard, in the Hudson River Valley Area. One can see that the hazard level curve used by AASHTO for a 500 year return period is very different from the more recent USGS curves.

Current seismic study guidelines seem to favor a two level approach, using both 500 and 2500 year return periods, with difference performance criteria for the two different return periods. (Note that the 500 and 475 year periods and 2475 and 2500 year return periods are the same.) This approach is discussed in ATC-18 Report, Seismic Design Criteria for Bridges and other Highway Structures.

Perhaps at best one can conclude that for a movable bridge one needs to deal with each bridge on a case-by-case basis, particularly in evaluating an existing bridge.

There are several unique aspects associated with the seismic performance of a movable bridge. First is that there is typically a large mass of counterweight, which is part of the bridge superstructure. And typically the counterweight is at least as heavy as the span it balances thus increasing the weight of the superstructure to at least twice that of a typical fixed bridge. Second the supports for the movable span are machinery elements such as trunnions, bearings, pivots, rollers, tracks. wedges, lock bars and guides that are designed to resist dead, live and possibly wind loads with no capacity for uplift. And these are typically carefully machined and aligned elements that have little if any tolerance for misalignment, particularly if the bridge is to move. Third, the bridge does not carry live load when open so it may be less critical in that position. Fourth, there are two structure types, one in the open position and one in the closed position. And fifth, for movable railroad bridges, there is no continuity of the rails and guard rails at the joints.

There is also a question about the appropriateness of using one half the seismic force for a bridge that is in one position 10% of the time and the other position 90% of the time.

If we consider this proposed 90% limit it means that in 24 hours, a bridge that is normally closed, is open 2 hours and 24 minutes. Allowing each opening to be 6 minutes, a bridge would need to open 24 times to meet the 90% rule. For a 50-year time span (often used in seismic hazard analysis for bridges and buildings), the bridge will be open for 5 years. Perhaps it might be better to calculate the various risk levels that would be associated with various ranges of open or closed.

Shown below is a table of normalized peak ground acceleration (normalized with respect to the 500-year value) for a normally closed movable bridge in the open position. The normalized peak ground acceleration values were derived using the criterion that the peak ground acceleration has a 90% probability of not being exceeded in the given exposed time.

Open Time		Return Period	Normalized Peak Ground Acceleration	
		Years	New York	San Francisco
0%	(0 yr.)	0	0	0
10%	(5 yrs.)	50	0.11	0.45
20%	(10 yrs.)	100	0.22	0.57
30%	(15 yrs.)	150	0.34	0.67
40%	(20 yrs.)	200	0.45	0.75
50%	(25 yrs.)	250	0.55	0.80
60%	(30 yrs.)	300	0.65	0.85
70%	(35 yrs.)	350	0.75	0.90
80%	(40 yrs.)	400	0.85	0.95
90%	(45 yrs.)	450	0.93	0.98
100%	(50 yrs.)	500	1.0	1.0

Thus the appropriate acceleration level is a function of both the exposure time and bridge location. Based on the data presented above, it appears that using one half the seismic force for a bridge in the open position is reasonable in San Francisco, California (i.e., for 10% exposed time, the normalized value is 0.45). However, if the same criterion is used for bridges in New York, it may lead to very conservative results.

As a movable bridge is always built over a waterway, scour is almost always a possibility and its effect on the seismic response and vulnerability needs to be considered. (The same question arises for the case of ship collision.) And the question is how much scour should one consider in combination with a seismic event? There are no code requirements although a paper presented by Knott at the 1997 Extreme Load Events and Their Combinations, Conference Proceedings, Design of Bridges for Extreme Events, December 1996, Atlanta gave some guidance on this.

Thus if one is dealing with an important movable bridge there are many possible load cases.

Case	Return Period	Scour	Open/Closed
1	500 yr.	No	Open
2	500 yr.	Yes	Open
3	500 yr.	No	Closed
4	500 yr.	Yes	Closed
5	2500 yr.	No	Open
6	2500 yr.	Yes	Open
7	2500 yr.	No	Closed
8	2500 yr.	Yes	Closed

And even if we can agree on the load cases we need to also consider what performance criteria are appropriate. The current New York State Performance Criteria for Critical Bridges is attached. For a 500 year return period, the bridge is to have minimal repairable damage. For a 2500 year return period, the bridge is to be available for limited emergency use within a 48 hour period or repairable to full service within months. While there are various interpretations as to how one is to apply this to the bridge superstructure and substructure, typically expressed in terms of Demand/Capacity ratios, how is this to be applied to the bridge machinery?

This is an issue that is still under consideration but some suggestions are as follows.

For a 500 year return period all machinery should remain in the elastic range, i.e. there will be no permanent deformations. In addition all machinery should remain in alignment, and should not be displaced.

For a 2500 year return period, all machinery that supports the movable span, such as the pivots, rollers, track and end lifts for a swing span trunnions or tracks for a bascule span, and sheaves and guide for a lift span can experience some plastic deformation but must remain essentially intact. However, the operating machinery such as the rack, pinions, shafts and bearings could be damaged. It is also suggested that if the operating machinery could be damaged then one should provide means to operate the bridge using a temporary winch and wire rope system or similar arrangement. The intent is that the bridge could be repaired and returned to service.

But it is also suggested that one needs to apply these suggestions on a case by case basis. There are some bridges that would need to be returned to operation quickly, and others where returning to operation is not critical.

In some cases this may lead to unacceptable conditions for the machinery and it is suggested that an alternative may be to design the bridge supporting structure to help isolate the mechanical elements. This is along the lines used by hospitals in critical seismic regions which are built on base isolation devices.

An example is the West Seattle Swing Bridge, a low-level bridge hydraulically operated, double-leaf concrete swing bridge. Among its engineering and structural innovations are the concrete box leaf design, the movable hydraulic system, and the high-strength concrete. The hydraulic system lifts and rotates 7,500-ton leaves in open and closed positions. Given the large mass and seismic demands of the Seattle, Washington area, the design incorporated a pile within a pipe sleeve foundation system to allow the bridge structure to be articulated. This foundation system functioned to increase the natural period of the structure, hence reducing the seismic demand.

Needless to say much needs to be done, and it is hoped that these comments will lead to some discussion and perhaps guidelines that will be of use to our industry. Solutions are available. for example one could replace the machinery, but this is an expensive proposition. Alternatively one can add sufficient restrains so that the swing span will remain in place. But there are other solutions available, at least for new bridges that do not involve the machinery. These have to do with how the structure is supported.

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