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The Effect of Mounting Location on Traffic Induced Vibrations of Bridge Mounted Overhead Sign Bridges

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The University of Texas at Austin
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conducted for the

Texas Department of Transportation

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PHIL M. FERGUSON STRUCTURAL ENGINEERING LABORATORY

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Chapter 1

Introduction

The previous study of overhead sign bridge (OSB) vibrations showed that the vibrations were caused by the vehicle induced accelerations of the bridge and that the OSB vibrations could be predicted analytically buy using the measured bridge accelerations as input to a dynamic structural model of the OSB. Thus, bridge acceleration records collected on a bridge at a variety of assumed mounting locations can be used to analytically study the expected dynamic response of OSBs at the various mounting locations. This methodology is used in this investigation is to determine OSB mounting locations for which vibrations are not excessive.

The effect of mounting location on the vibration was studied by measuring the accelerations of four (4) bridges along half of their span. The accelerations were measured at 1/8 points to mid span on both exterior girders. The accelerations measured were due trucks traversing the bridge. Thirty (30) or more acceleration records were recorded on each bridge. The largest acceleration record for each bridge was used to excite a finite element model of the overhead sign bridge (OSB). The peak displacements from the dynamic analysis were used to characterize the response of the OSB to the measured accelerations.

All of the bridges included in the study were composite prestressed “I” girder bridges. Their span length varied from 106 to 130 feet. The test bridges were selected to cover the range of commonly used bridges in Texas.

The analytical study included the influence of OSB span and the type of sign. The OSB span lengths were 50, 75, 100, and 120 feet. The OSB dimensions were taken from TxDOT standard truss type sign bridge design standards. The acceleration records from fourt (4) sections along the span of the 4 bridges were used to excite the models of the four (4) span lengths OSB with four (4) different sign configurations. The sign configuration included no sign, standard sign 20 wide, LED sign, and fiber optic sign. A total of 4x4x4x4=256 separate dynamic analyses for were performed in the study.

Chapter 2 describes the field measurements and observations of the bridges response. Chapter 3 gives the details of the dynamic finite element analysis of the OSB when excited by the selected acceleration records from each bridge. The analysis of the response is also analyzed and summarized in this chapter. The last chapter gives the summary of the study and the recommendations. The appendices contain the detailed data used in the study.

Chapter 2

Field Test Results

2.1 Overview of Field Tests

Four bridges were instrumented to measure the magnitude of the traffic induced vibrations along the span. These field measured accelerations were then used as input for the FEA models of the sign bridges. The field tests were conducted during the period from 6/26/2006 to 11/21/06. The four precast concrete girder bridges were chosen to include a variety of span lengths, widths, support conditions, and skews. All bridge sites are located in Austin, Texas. The locations of the selected bridge spans are listed below.

Site 1: US290/SH71 (BenWhite Blvd. near Victory Lane) left main lanes span 29. The overhead sign bridge "OSB-W" which was one of the subjects of the previous investigation is mounted on this span. This span was selected because it has a known OSB vibration problem and high traffic loads.

Site 2: US290/SH71 left main lanes span 30. The span adjacent to site 1 was selected for a different span length and high traffic loads.

Site 3: Loop 1 at Barton Creek right main lanes. This is the longest span studied.

Site 4: US183 near Burnet Road left main lanes. This is a short bridge with a 20 degree skew.

The geometry of the test bridges is shown in Table 2.1.

Table 2.1 Test Bridge Geometries

Site	1	2	3	4
Location	Ben White Span 29	Ben White Span 30	Loop 1 /Barton Creek	183/Burnet
Span-feet	110'	120'	130'	106'
Width/Lanes-feet	70'/4	70'/4	52'/3	70'/4
Skew (degrees)	0	0	0	20
Traffic Dir	W	W	N	N
Entry/Exit	Exit	Exit	Exit	Entry
End Condition	Cont/exp	Cont/Cont	Exp/exp	Cont/cont
Number of Accelerometers	8	8	8	10
Number of Inclinometers	0	4	4	2

Each field test was conducted by installing accelerometers and inclinometers at locations along the span on underside of the outer girders. Vertical accelerations and rotations about the transverse axis of the bridge were collected using a National Instruments data acquisition system sampling at a 500 Hz rate. Data records of 20 to 25 seconds in duration were made by triggering on high-level acceleration events corresponding to passage of heavy trucks. More than 30 data records were recorded for each site.

2.2 Sensor Layout

Figures 2.1 through 2.4 illustrate the sensor locations at each of the four sites. Accelerometers were installed at each of the numbered stations on the underside of the two outer girders at positions of 1/2, 3/8, 1/4, and 1/8 of the span length. The accelerometers were oriented to sense vertical acceleration. The outer girders were monitored since these would be the girders that would be used to support a sign bridge.

The instrumentation layout of site 4 (Figure 2.4) is unique due to the bridge skew. The accelerometers were located at the same fractions of span along the right girder, paired with accelerometers perpendicularly opposite along the left girder. These locations represent the hypothetical bases of an OSB since an OSB would be positioned perpendicular to the long axis of the bridge, not parallel to the skewed pier.

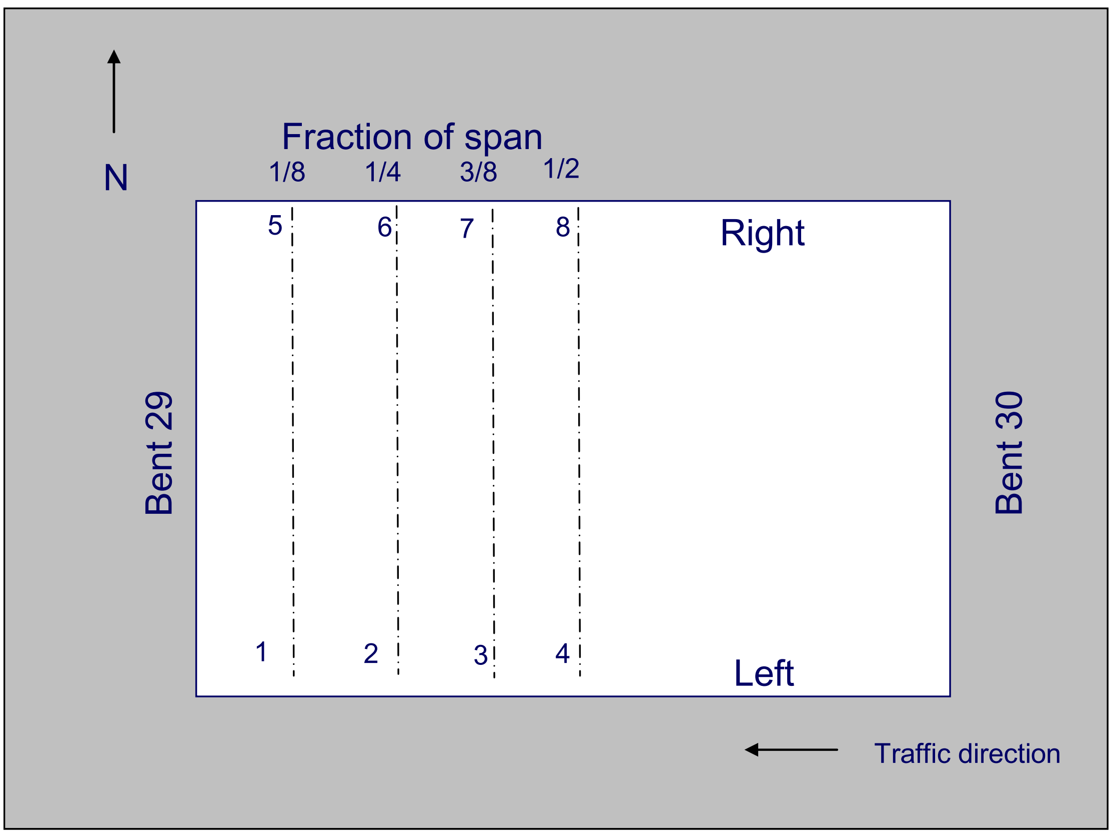


Figure 2.1 Instrument Location Site 1: Ben White Span 29

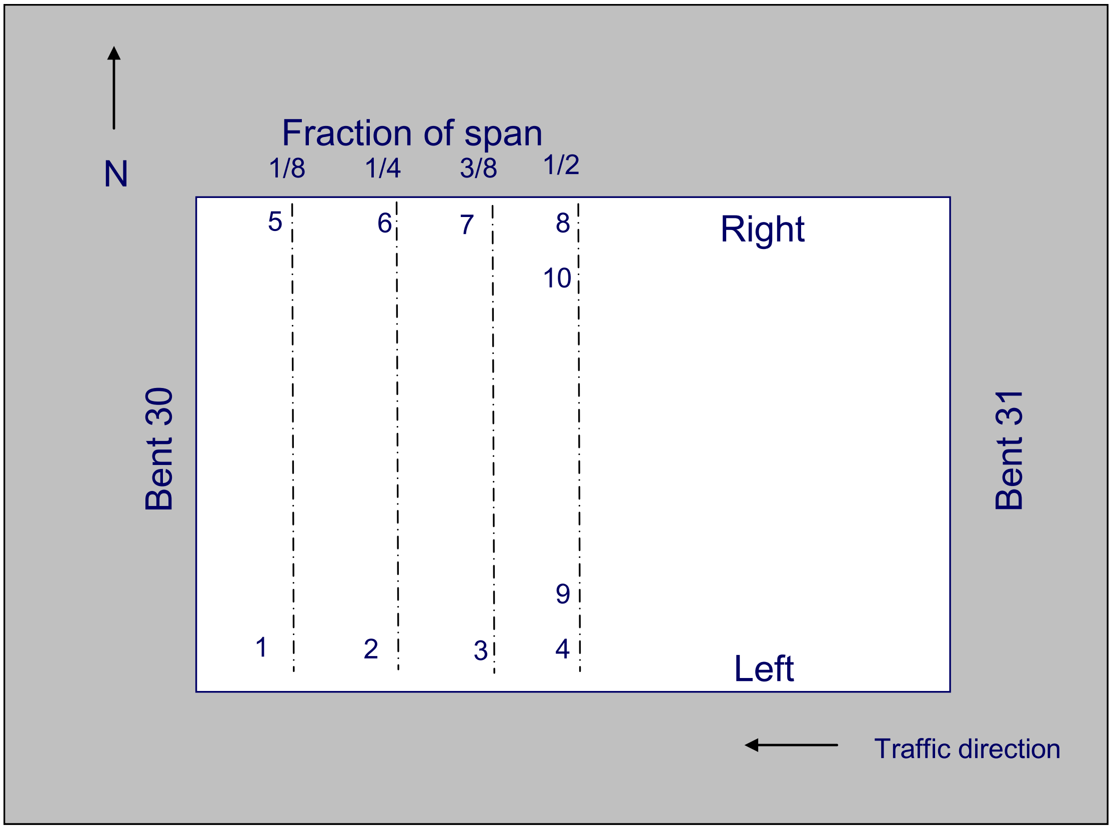


Figure 2.2 Instrument Location Site 2: Ben White Span 30

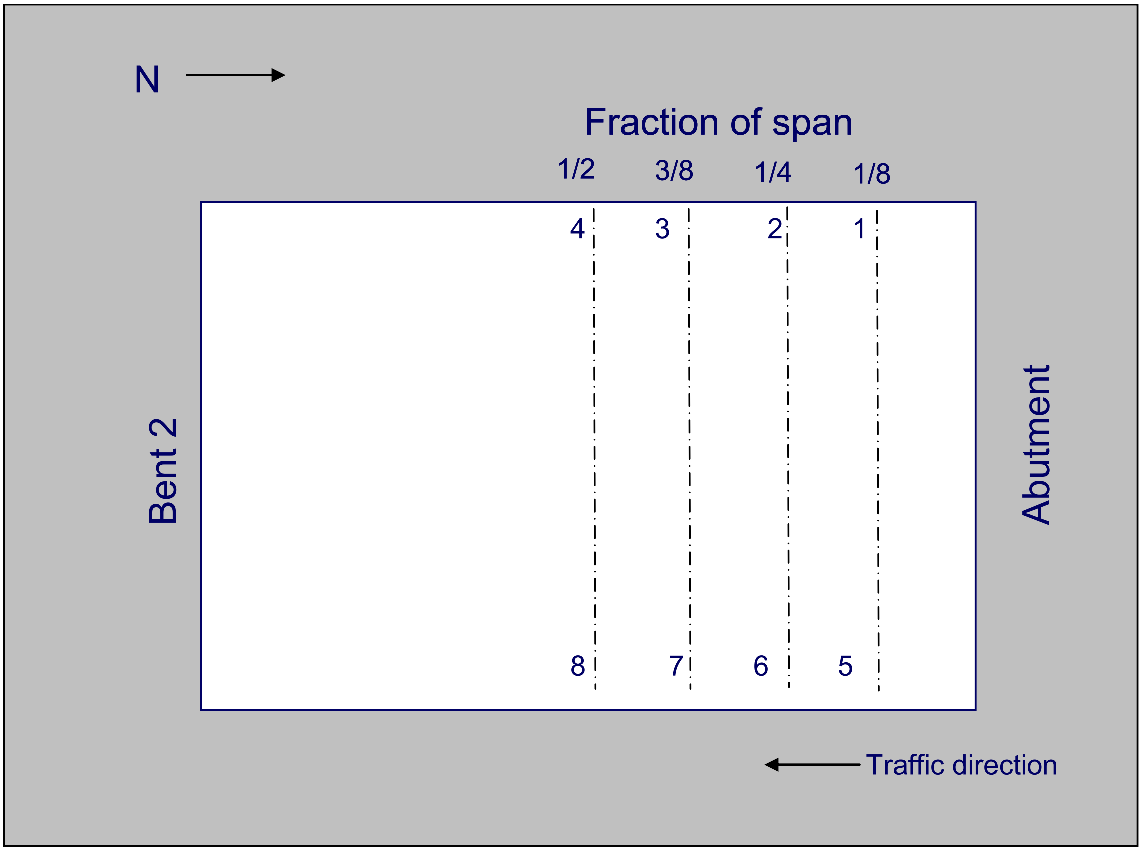


Figure 2.3 Instrument Location Site 3: Loop 1 at Barton Creek

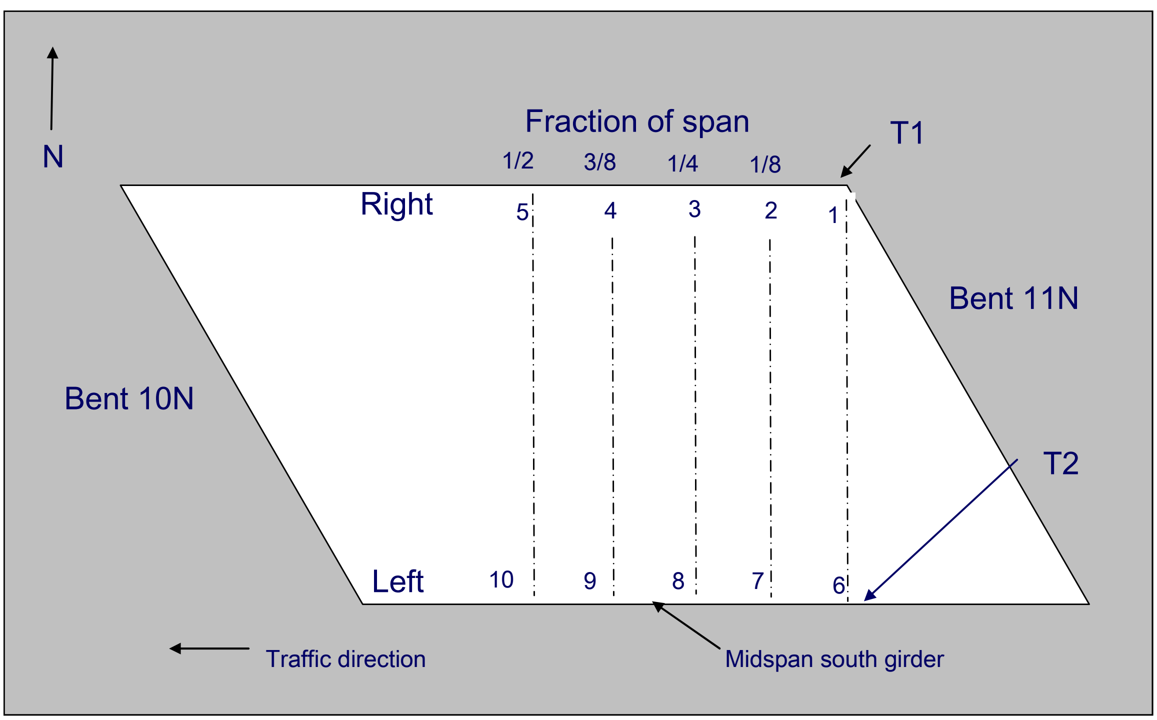


Figure 2.4 Instrument Location Site 4: US 183 and Burnet Rd.

2.3 Data Analysis

In addition to providing acceleration data as input to the FEA model, the field data was analyzed to determine the frequency response and the dynamic deflected shape of the bridges from the displacements computed from the measured accelerations.

2.3.1 Frequency Response

Fig. 2.5 shows a typical large-level acceleration event (truck passage) for the first Ben White span, site 1, as a time-series and a frequency domain representation. The acceleration record shows the typical sudden onset of vibration due to one or more trucks crossing the bridge and the slow decay of the vibration. The spectral analysis shows two strong peaks in the 3 to 4 Hz range. These results are typical of all four bridge spans studied. The peak frequency response varied from 2.5 to 4 Hz depending on the span length. The summary of the fundamental frequencies for the four bridges is given in Table 2.2.

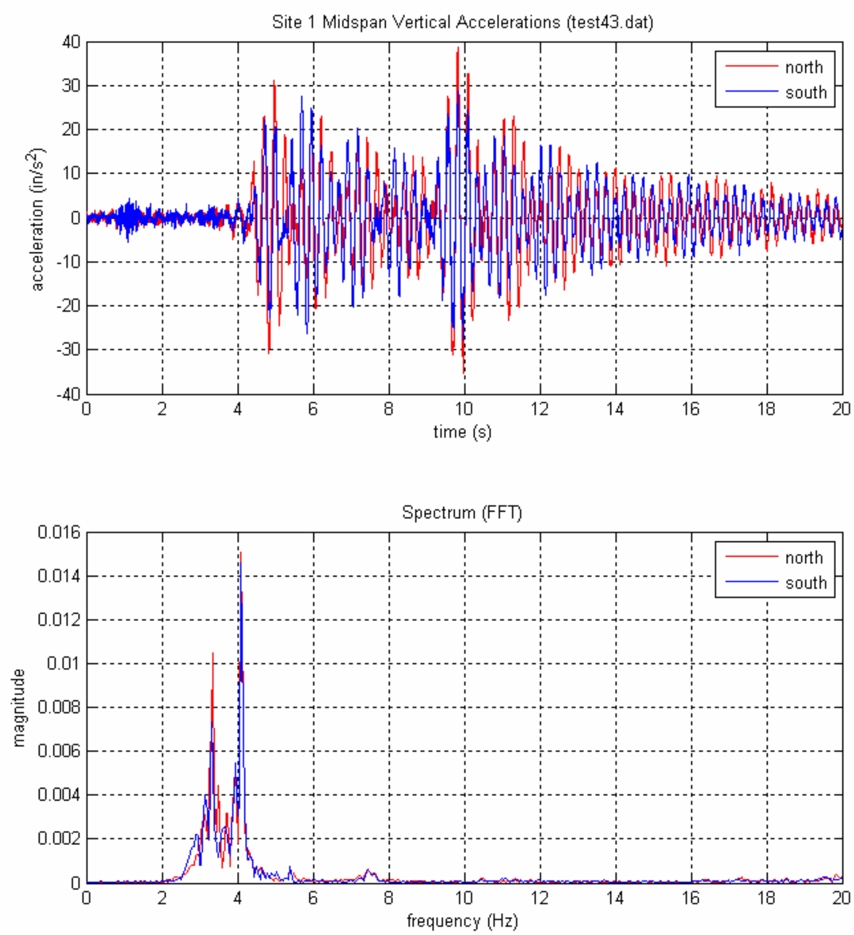


Figure 2.5 Site 1 Acceleration Event and Spectral Analysis

Table 2.2 Summary of Fundamental Frequencies

Site	Span (ft)	f0 (Hz)	f1 (Hz)
1 Ben White 29	110	3.3	4.1
2 Ben White 30	120	2.8	3.7
3 Loop 1	130	2.5	3.9
4 US183	106	3.9	4.0

Figure 2.6 shows the frequency response of site 1 at each accelerometer location. The figure is laid out with the plots on the left side representing the left (south) side of the roadway, and the right-hand plots represent the right (north) side of the roadway. The plots are ordered top to bottom on the page from the locations closest to the pier to the mid-span locations. It can be seen from these plots that the strength of the accelerations recorded is highest at the midspan and decreases towards the pier.

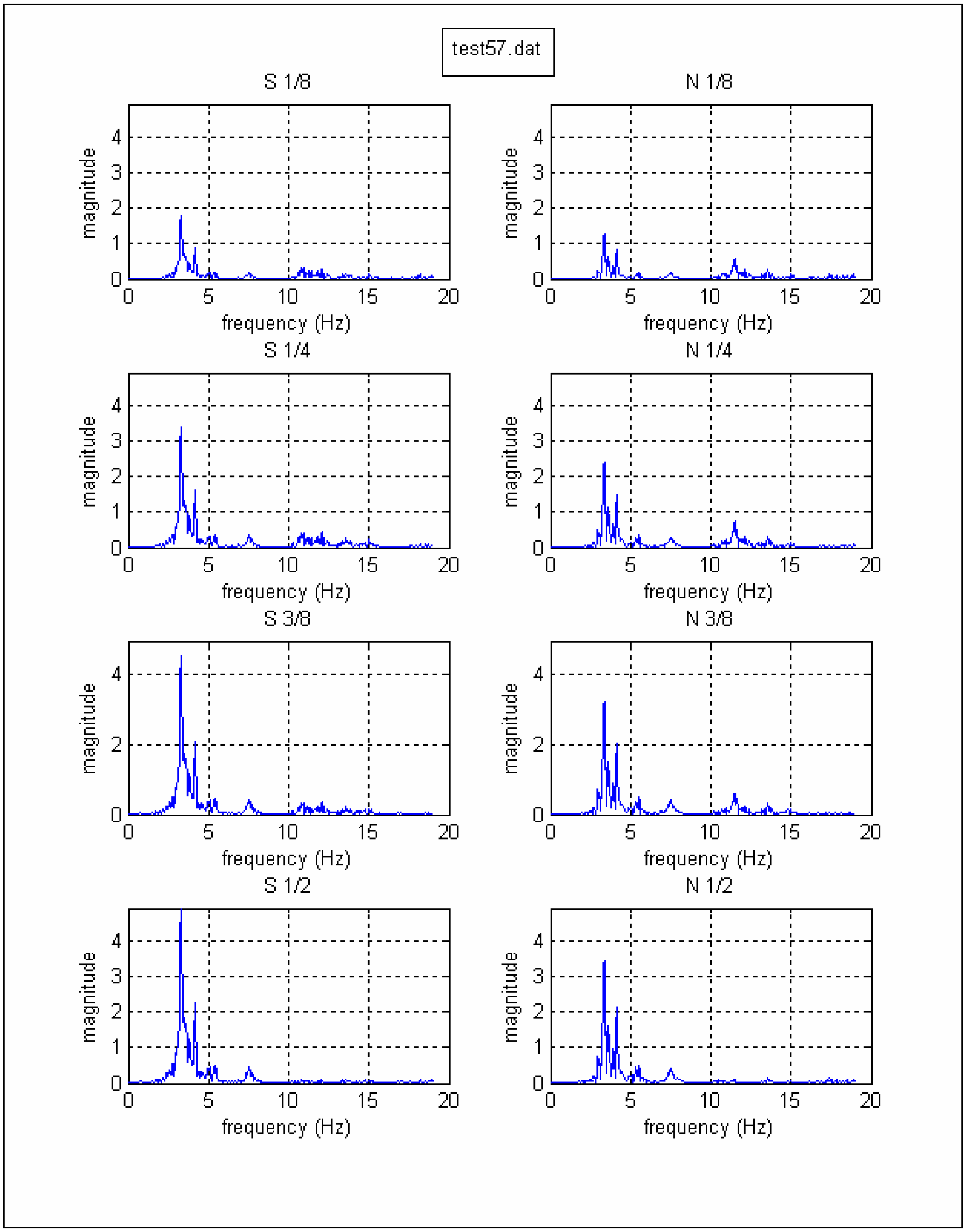


Figure 2.6 Site 1 Frequency Responses at Each Accelerometer Location

2.3.2 Dynamic Deflected Shape

The dynamic deflection of the bridge deck was computed for each acceleration record for each site integrating the acceleration twice. In order to estimate the deflected shape of the bridge, the RMS values of the computed deflection at each sensor station along the bridge were compared. As can be seen in Figure 2.7, the relative strength of the RMS displacement response along the length of the span corresponds well with a half-sine shape. The half-sine shape is the expected deflected shape for a simply-supported beam vibrating in its fundamental flexural mode. The same result was found for each of the bridges instrumented.

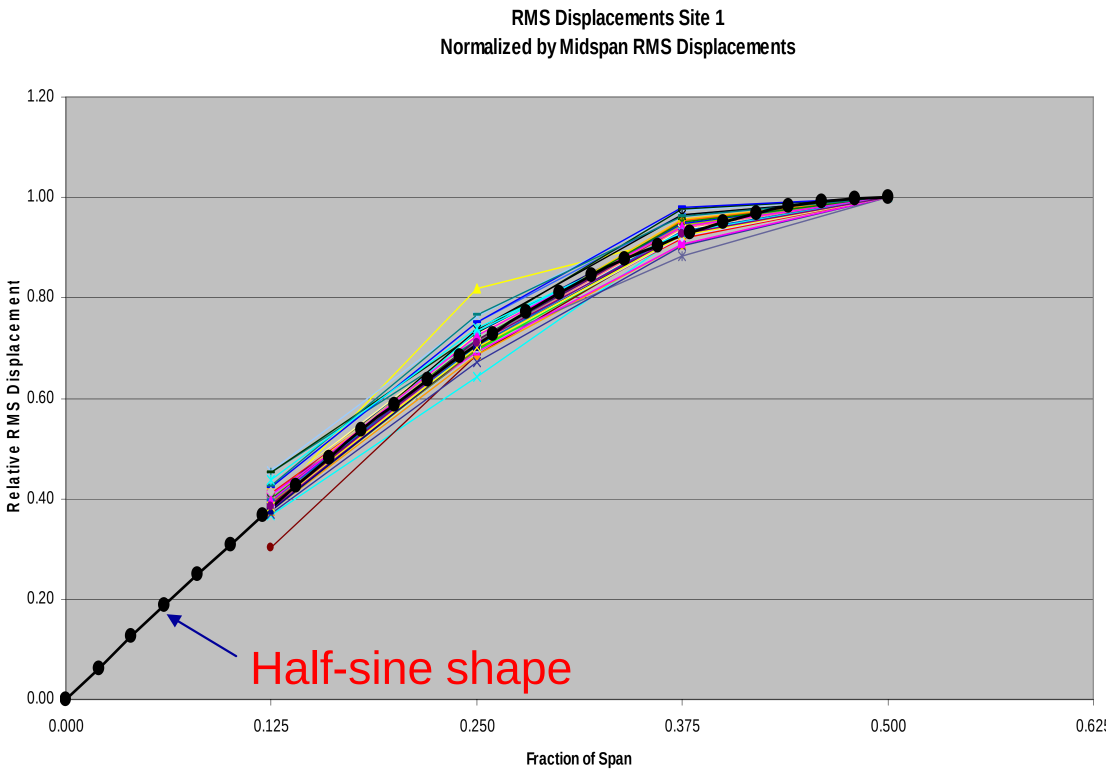


Figure 2.7 Site 1 Normalized RMS Displacements Compared to Half-Sine Shape

2.3.3 Visualization of Computed Deflections

As part of the data analysis, a 3-D animation of the computed displacements was created for two of the data records recorded for site 1. The visualization clearly illustrates the complex nature of the bridge deck motion. The bridge responds to different dynamic loads—one or more trucks passing in various lane configurations—by vibrating in a mainly flexural mode (both edges of the bridge moving in phase), a strongly torsional mode (the bridge twists about its long axis), and combinations of the two.

While it is not possible to incorporate the animation in this report, the following Figures 2.8 through 2.10 illustrate these phenomena. Figure 2.8 shows the deflected shape of the bridge vibrating in a strongly bending mode. In the figure the bridge is cut at mid-span and the upper left edge represents the pier, which is not moving. The upper and lower surfaces plotted are the positions of the bridge deck at the upper and lower peak of one displacement cycle. The middle surface in the plot represents the undeflected shape. As can be seen from the plot, both the left and right sides of the bridge (the lower left and upper right edges of the plots) are moving together in-phase.

In contrast, Figures 2.9 and 2.10 show the bridge deck moving in out-of-phase motion. Figure 2.9 illustrates a cycle where there is large motion at the right edge of the bridge while the left edge is fairly still. Figure 2.10 is several seconds later in the same data record and shows the left edge moving strongly while the right edge is not.

Note that the vertical displacements are exaggerated in the plots. The calculated dynamic displacements were typically no more than 0.2” peak-to-peak.

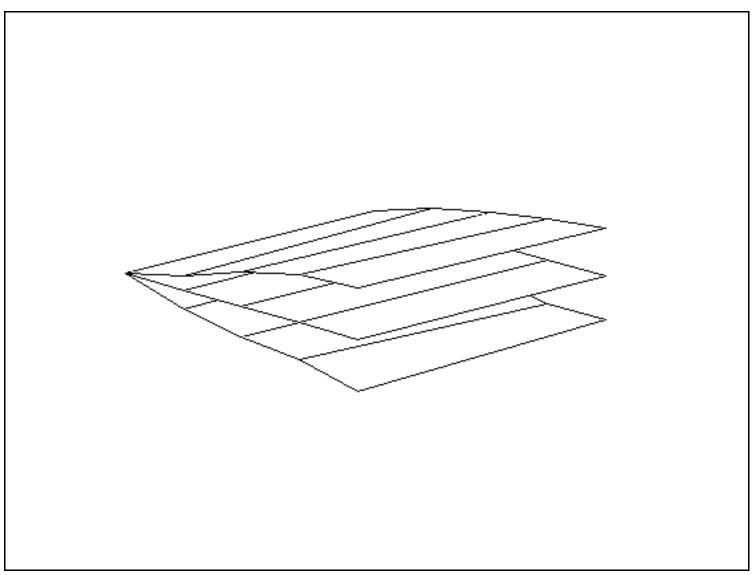


Figure 2.8 Deflected Shape Predominantly Bending (In-phase motion)

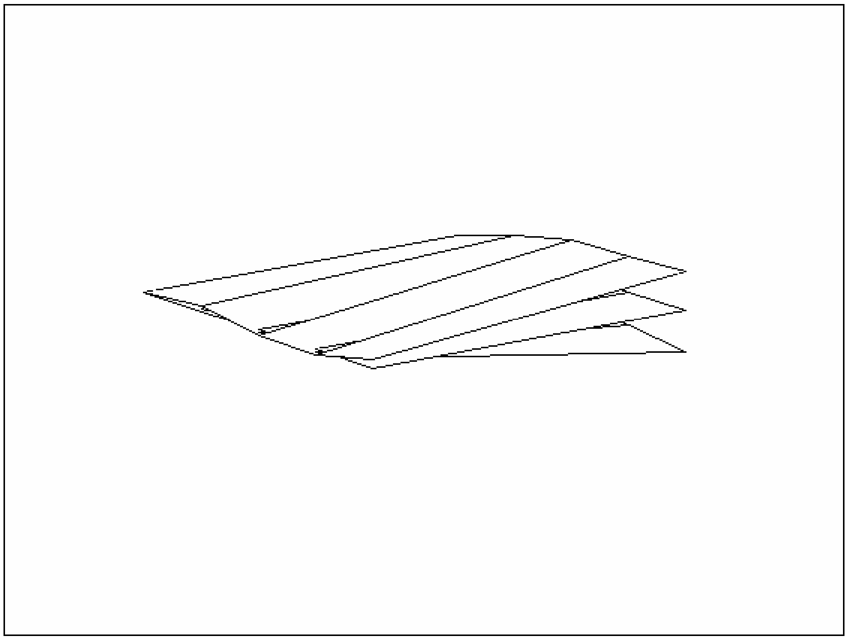


Figure 2.9 Deflected Shape Predominantly Torsion (Out-of-phase motion)

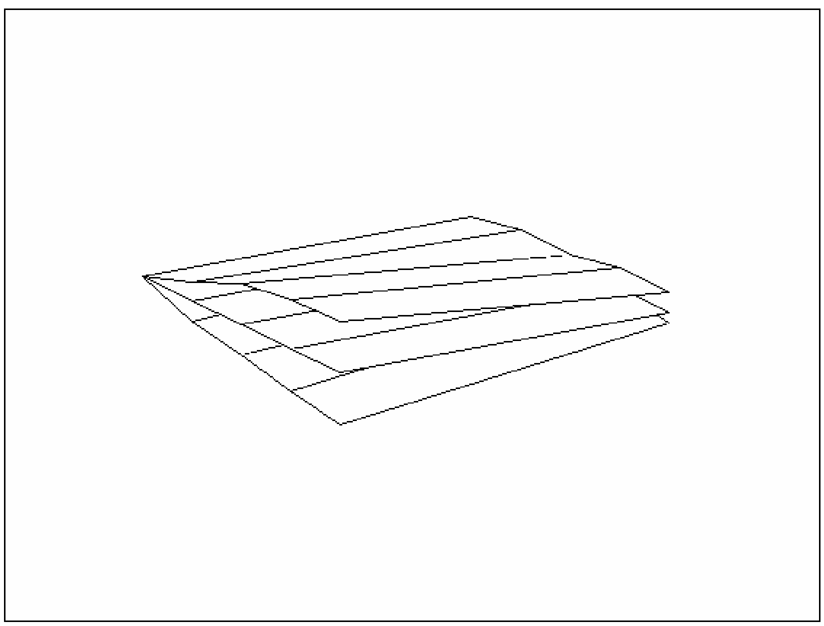


Figure 2.10 Deflected Shape Predominantly Torsion (Out-of-phase motion)

2.3.4 Effect of Bridge Skew

Figure 2.11 plots the relative displacement of the two sides of the skewed bridge at site 4, US183 near Burnet Road. Because on a skewed bridge an OSB would be mounted perpendicular to the roadway (not parallel to the pier), the two ends of the OSB are located at different positions along the two outside girders. For example, the site 4 bridge has a 20-degree skew. An OSB mounted with its left tower at midspan of the left girder would have its right tower mounted between the 1/4 and 3/8 span of the right girder.

The data in Figure 2.11 was computed by taking the RMS value of the computed displacements for each of the 41 data records at each position along the span for each girder and normalizing by the 3/8 span value of the left girder. The result is an RMS displacement value for each of the six locations of interest for each data set. The results were then averaged across the data sets.

The resulting plot shows that at each position along the bridge span, the two OSB towers are subjected to significantly different vertical displacements. At the nominal 1/8 span location, the average displacements on the left side were 50% higher than on the right side.

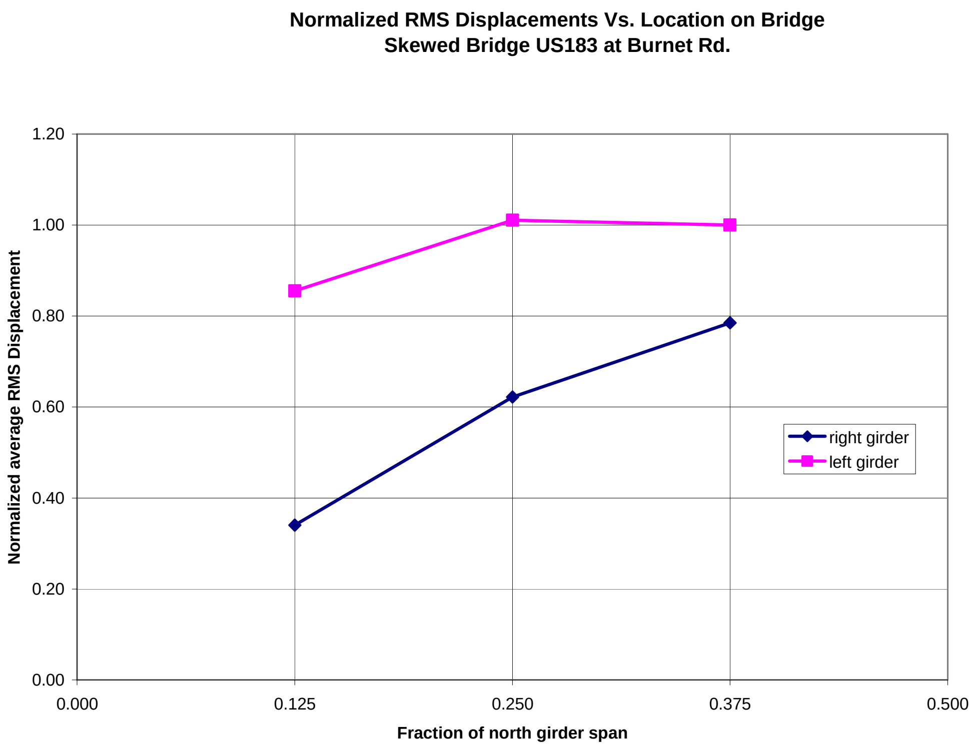


Figure 2.11 Relative Displacement of Left and Right Sides of Skewed Bridge

Chapter 3

Numerical Simulation of the Dynamic Responses on Overhead Sign Bridges

Numerical simulations of the overhead sign bridges (OSB) were undertaken to investigate the dynamic displacement of the sign structures when excited by the bridge motions measured in the field. The general purpose finite element program ABAQUS was used in this analysis. For each of bridges tested in the field, the effect of the mounting position of the sign along the span, the span of the sign bridge, and the type of sign on the sign bridge was investigated.

3.1 Modeling Parameters

3.1.1 Geometries and Member Properties of OSB Truss

The overhead sign bridge (OSB) consists of two main parts; the truss structure and the sign. The Texas DOT standard drawings were used to model the truss structure. The height of the truss structure was assumed as 280 in for all models. Four different OSB span lengths (50', 75', 100', and 120') were investigated. Figure 3.1 illustrates one of the models used for the dynamic displacement response simulation. Details of member size used this simulation given appendix A.

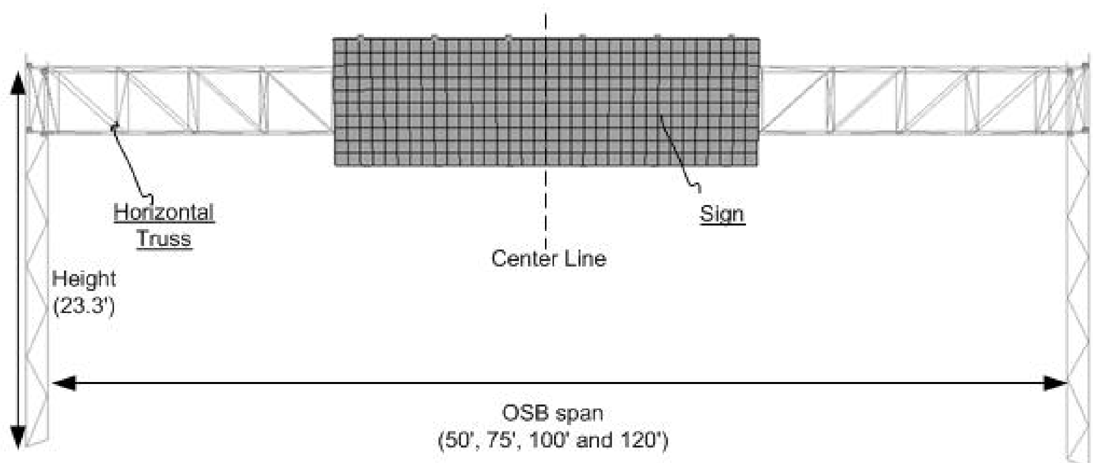


Figure 3.1 Typical OSB and sign

3.1.2 Sign Types

Four different sign types and conditions were included in this simulation.

1. No Sign – bare OSB
2. Regular Sign – standard sign with walkway and lighting in front
3. Fiber Optic Sign (FOS) – variable message fiber optic sign

4. LED Sign – variable message LED sign

These are shown schematically in the Figure 3.2. The no sign condition was included to provide a base condition in order to determine the influence of the sign mass and its location upon the response of structure. All signs were assumed to be mounted on the center of the horizontal truss of the OSB.

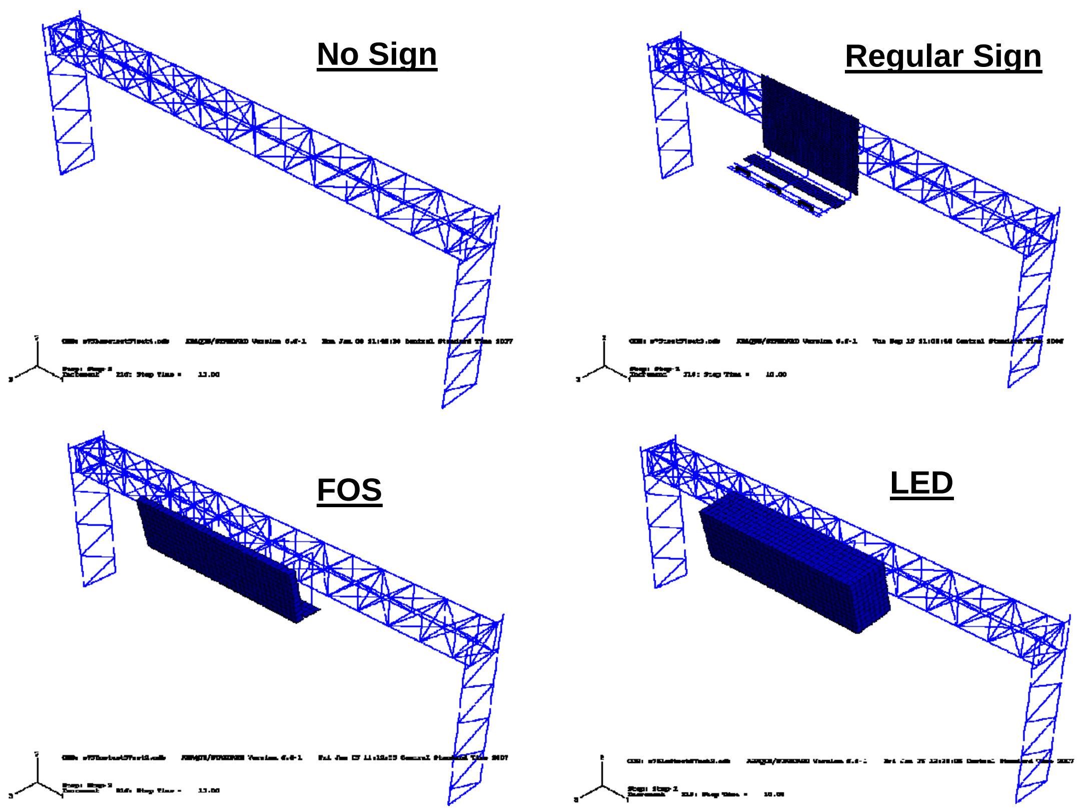


Figure 3.2 Sign types mounted on the base truss structure

The actual lateral location of the signs on the OSB and the size of the signs vary. The simulation was limited to the typical sign configurations in use which were provided by TxDOT. The sign configurations studied are illustrated in the figure 3.3. The weight of the signs is also listed in the figure. The estimated weights and mounting orientation were provided by TxDOT personnel

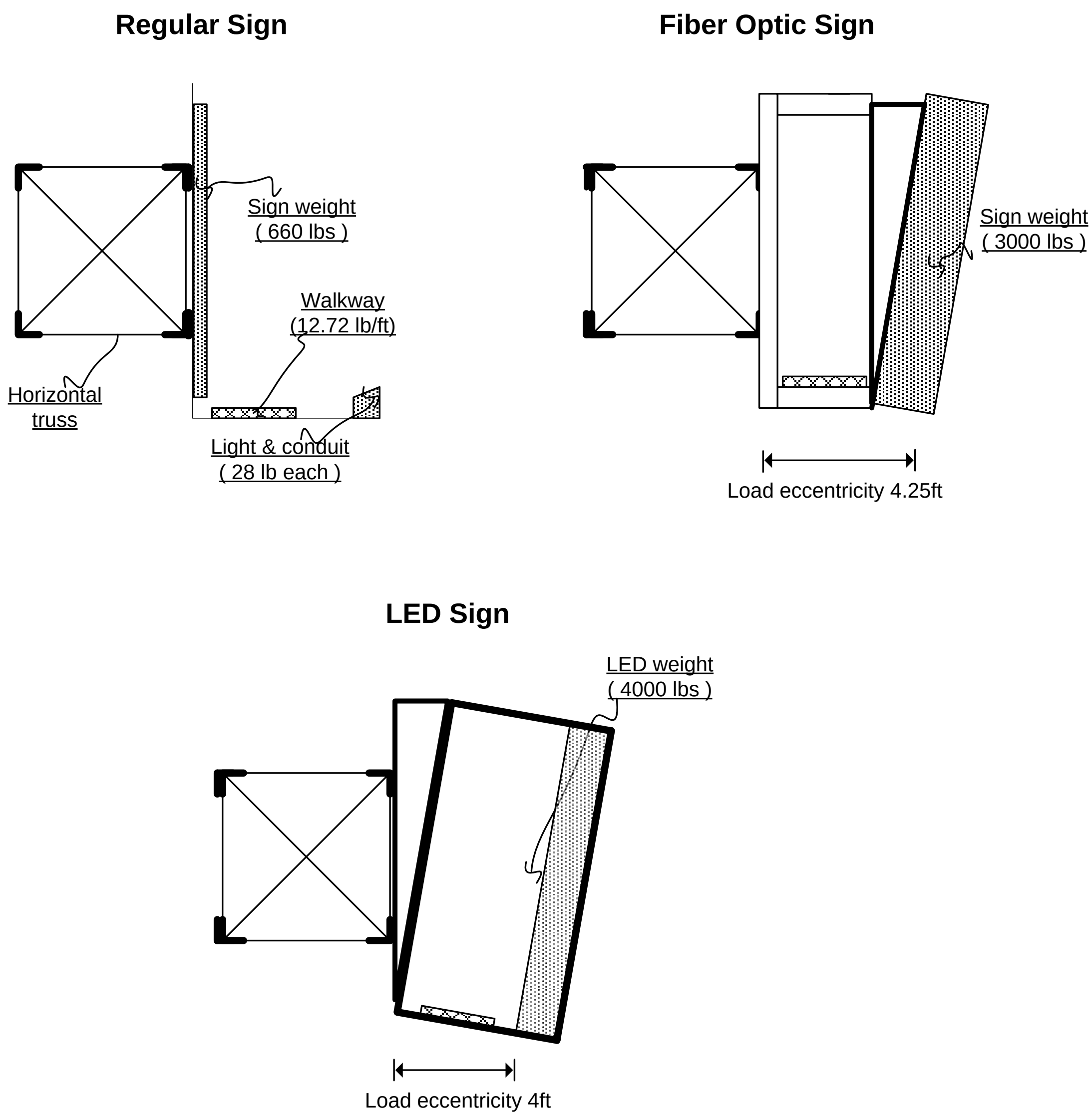
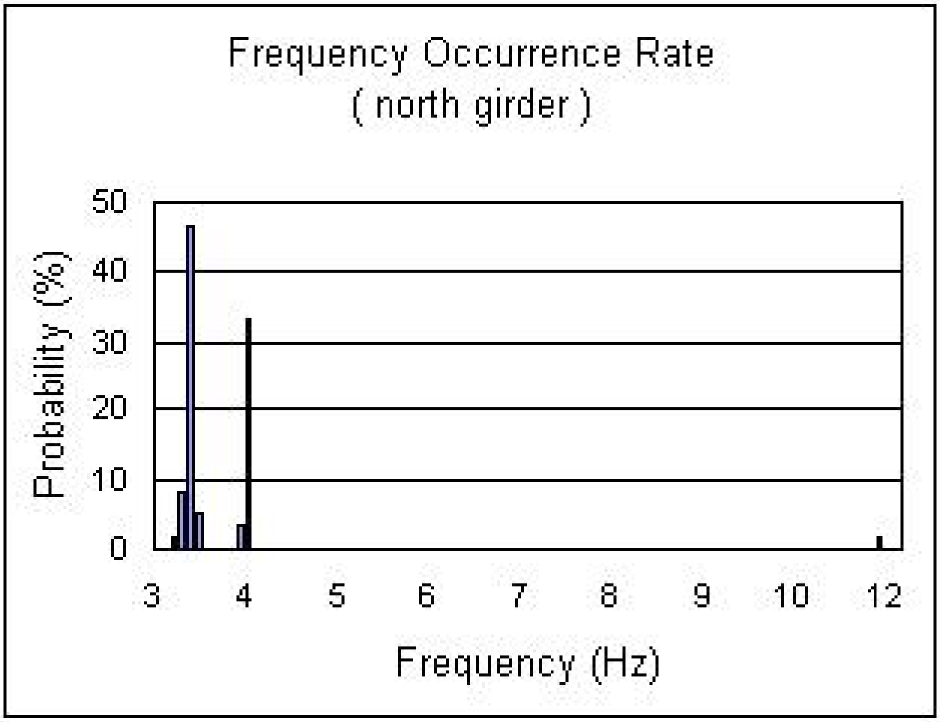


Figure 3.3 Schematic View of Signs Mounted on the Base Truss

3.2 Statistical Review of Input Accelerations

The acceleration data collected at each site was reviewed statistically to select representative data sets for the simulations. At the site 1, 60 acceleration data samples were obtained. Figure 3.4 shows the frequency occurrence rate of the data at this site. Dominant frequency range is from 3.1 Hz to 4.2 Hz, the first and the second mode of highway bridge girders are included in this range. The maximum amplitudes occurred in approximately 96% of 60 test sets in either the first or the second mode.



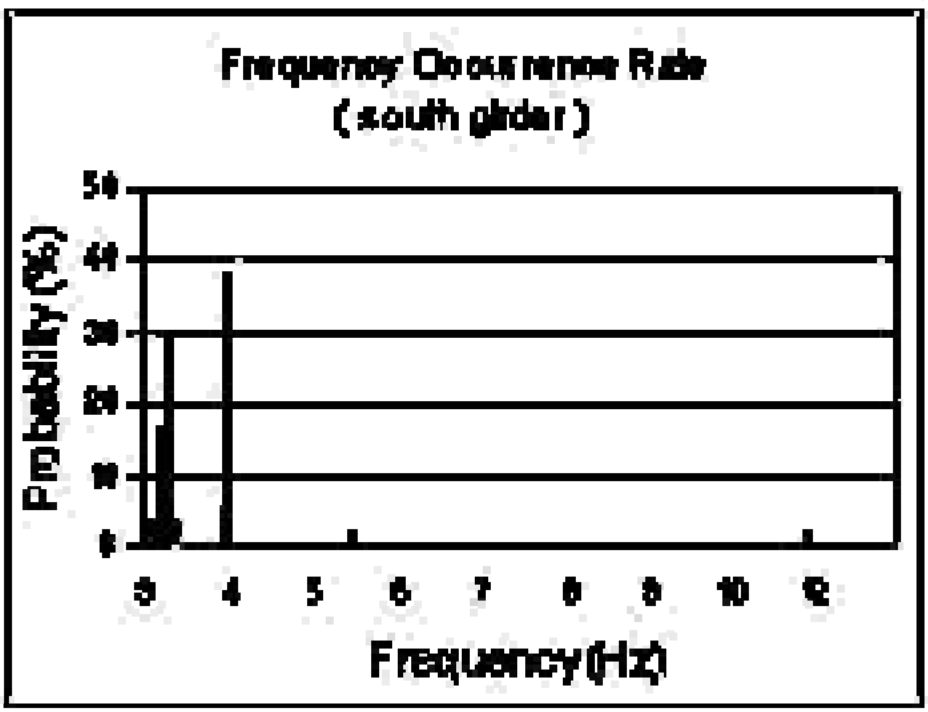


Figure 3.4 Frequency occurrence rate of each girder at the site 1

Test set 57 (one of test sets at the site 1) showed the largest amplitude in both the north and south girders in the field test data sets for this site. The north girder is the outmost girder on the right side and the south girder is on the left side looking in the traffic direction. The individual frequencies of the north and south girders fall within the 95% confidential interval of the samples which had maximum amplitude in the first mode. The distribution of the samples is presented in Figure 3.5. Therefore, Test set 57 was considered as representative of the acceleration behavior of the highway bridge at site 1 and was used as the input acceleration into the simulation models of site 1. At other sites, the same approach was used to select input accelerations to the FE models. A comparison of the selected records used for each site with the statistics for each site is shown in Table 3.1.

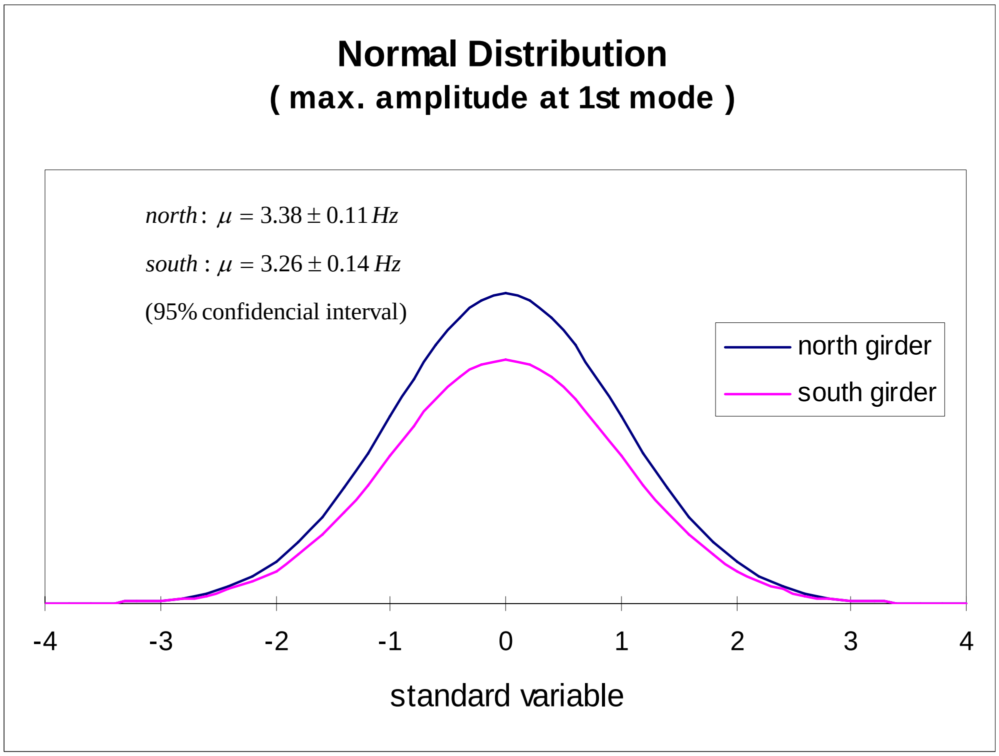


Figure 3.5 Normal Distribution of Samples

Table 3.1 Selected Acceleration Data Set for Simulations

Location	Number of samples	Selected test set	Frequency of selected set		95% confidential interval	
			Left girder	Right girder	Left girder	Right girder
Site 1	60	57	3.4	3.3	3.39 ±0.11	3.26 ±0.14
Site 2	50	18	2.85	3.73	2.85 ±0.09	3.78 ±0.05
Site 3	32	13	2.48	2.48	2.56 ±0.25	2.52 ±0.17
Site 4	41	32	3.95	3.95	4.02 ±0.19	3.98 ±0.17

3.3 Results of OSB Dynamic Response Simulations

3.3.1 Effect Mounting Location Upon OSB Response

The effect of mounting the OSB within the span of the bridge was investigated by imposing on the OSB models the accelerations that were measured at four locations along each side of the highway bridge span; 1/8, 2/8, 3/8, and 4/8 of the span from the end support. The measured accelerations were used as input forces to the truss columns supports of the l OSB. The results were then compared to determine the effect of location on the response of the sign structure.

Vertical and horizontal displacement responses at mid span of the OSB horizontal truss were investigated using the measured accelerations. Figure 3.6 shows a vertical displacement response of a 100' span OSB model, with no sign, excited by the acceleration of the mid-span of the highway bridge of site 1. Total maximum vertical displacement was 1.1 in. from the static position and the maximum horizontal displacement was 0.76 in. The maximum dynamic displacements (vertical and horizontal) were determined for the four virtual mounting locations, for each sign configuration, and OSB span length. The results for the case of no sign on the OSB are shown in figure 3.7.

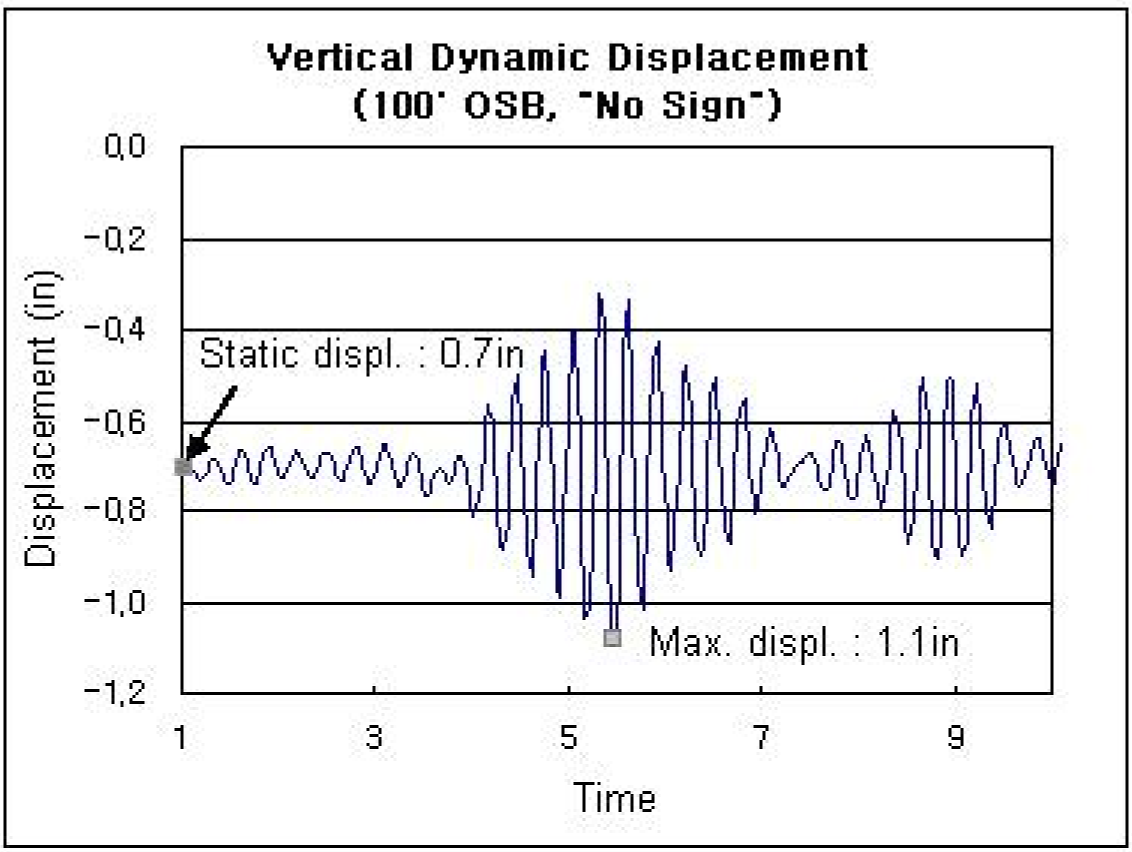
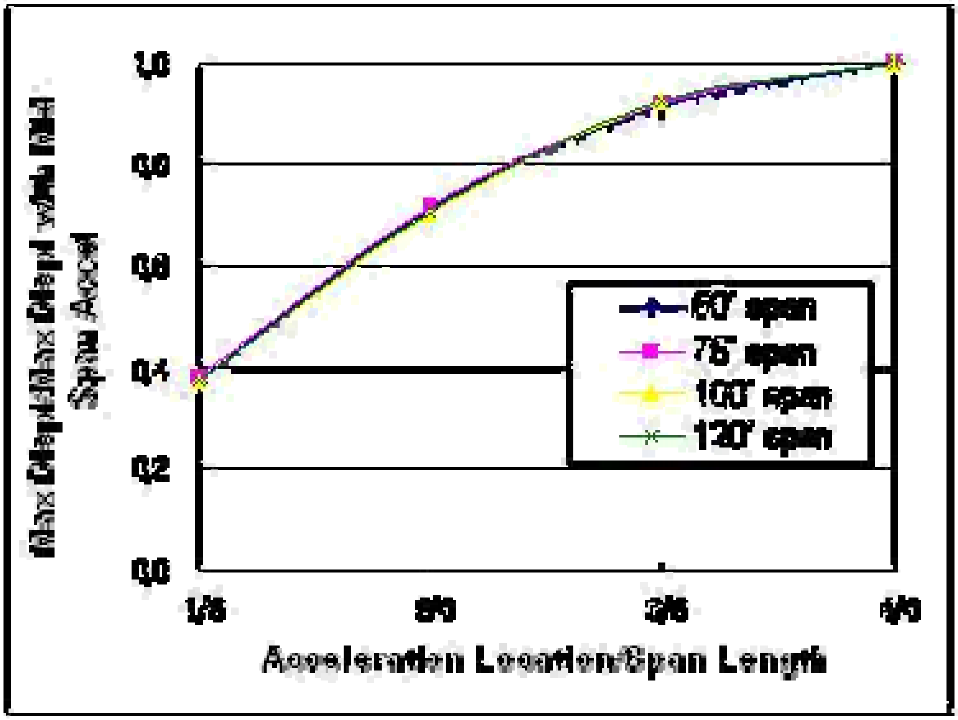
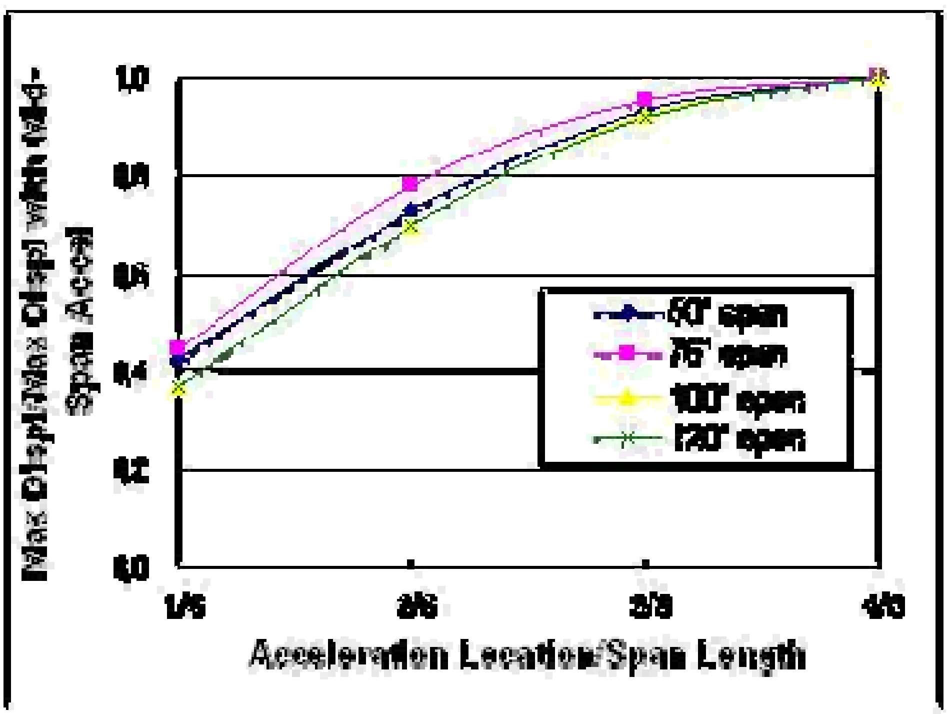


Figure 3.6 Vertical Displacement Response of 100' Span OSB.



a) Vertical Dynamic Displacement



(b) Horizontal Dynamic Displacement

Figure 3.7 Dynamic Displacement Variation (Site 1) No Sign

In Figure 3.7, maximum displacements of OSBs at each virtual mounting location are shown normalized by the maximum displacement of OSB of the same span length at the mid-span location of the highway bridge. The results show that vertical and horizontal dynamic displacements decrease by about 60% as the mounting locations are moved from mid span to the 1/8 span location near the end of the highway bridge. Furthermore, this tendency holds regardless of the OSB span lengths. A similar response reduction was also found when the three type of signs were mounted on the OSB. The displacement responses of various sign type of OSBs are presented in the appendix B.

The decrease in the dynamic responses follows from the decrease in the magnitude of accelerations along the locations presented in Chapter 2. The reduction of acceleration amplitudes implies the decline of input force intensity. Consequently, the decreasing input force results in reduced dynamic displacement behavior of the OSB. Therefore, the reduction of the OSB dynamic displacements as the mounting positions are moved toward the support should also occur with other types of signs. This reduction was confirmed for all sign types. The effect of mounting location along the span was similar for all sign type. The results for the other signs are shown in Appendix B.

3.3.2 Influence of Type of Sign Upon Dynamic Behavior

The dynamic displacement responses of various OSB models were compared with each other at the same mounting location. There are four different span length OSB models equipped with four types of signs, as described in the section 3.1. The results of OSB model responses on the 1/2 span of the highway bridge at site 1 are presented in Figure 3.8.

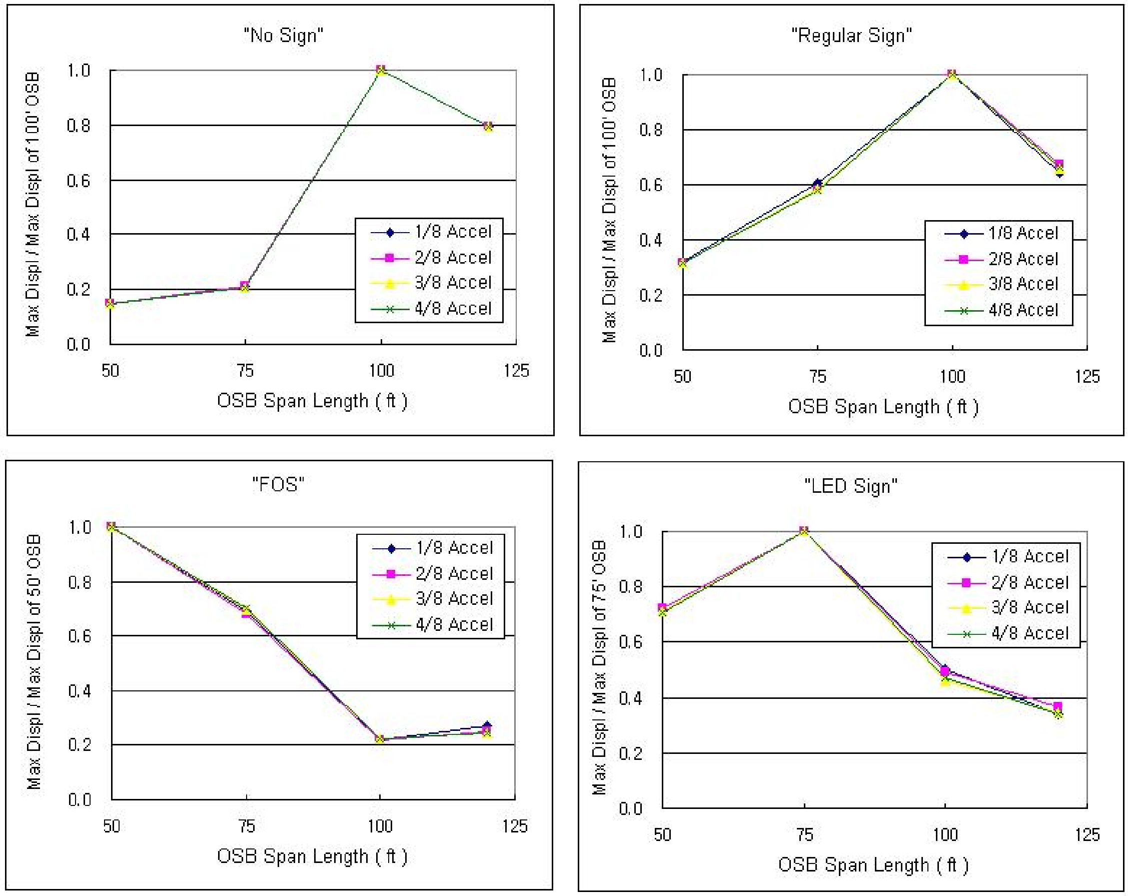


Figure 3.8 Vertical Dynamic Displacement Variations with Various OSB Spans (Site 1)

The graphs figure 3.8 indicate that the maximum response occurs at the 100 ft. span OSB for the “No Sign” and “Regular Sign”, while the maximum occurs at the 50 ft. span for the “Fiber Optic Sign” and at the 75 ft. span for the “LED Sign”. The OSB span length where the maximum displacement occurred varied for each type of OSB when different site accelerations were used. For instance, 125’ OSB span model showed the maximum displacement for the “Regular Sign” at site 2. Therefore, the OSB span length, type of sign mounted on the OSB, as well as the frequency of the bridge oscillations all affect the dynamic responses of OSBs.

3.3.3 Investigation of Displacement Spectrums

The natural mode and mode shapes of the OSB models were obtained and compared with the largest frequencies of the highway bridges in each site. Table 3.2 shows frequencies which have the largest amplitude in the each acceleration data set used for the FE simulations. The largest amplitude frequencies of highway bridges vary from 2.5Hz to 3.9Hz and match the first mode frequencies of the bridges. The variation of the frequencies is in inverse proportion to the span lengths of highway bridges. Three natural modes and mode shapes of each OSB model are shown in the table 3.3. The mode shapes of the OSB models are indicated as “T”, “H”, and “V”; the “T”, “H”, and “V” mean “transverse direction” of the traffic, “traffic direction”, and “vertical direction”. The combined mode shapes are designated by “V+H”, “V-H”, “H+V”, and so on. Example mode shapes are shown in the appendix C.

Table 3.2 Largest Amplitude Frequencies of Accelerations

Site Number	Location	Bridge span (ft)	1st freq. (Hz)	2nd freq. (Hz)	largest freq. (Hz)
1	Ben White 29	110	3.3	4.1	3.3
2	Ben White 30	120	2.8	3.7	2.8
3	Barton Creek (loop1)	130	2.5	3.9	2.5
4	US 183	106	3.9	4	3.9

The OSB modes were compared with the largest amplitude frequencies of the bridge accelerations. The proximity of the frequencies of the OSB model and highway bridge was investigated to explain the reason why the maximum dynamic displacements depend upon the OSB types. For instance, the largest mode of the highway bridge at site 1 is 3.3Hz. The regular sign shows its maximum vertical dynamic displacement at the 100 ft. span of the OSB at site 1 as presented in the Figure 3.8. When comparing the modes between the regular type OSB and the highway bridge, the closest mode of the OSB is the 3.35Hz of the 100ft span model. These results imply that the relatively larger responses of OSBs are induced by resonance phenomena between the OSB and the dominant mode of the highway bridge. The correlation between the largest amplitude mode of the highway bridge (fundamental mode) and the relevant OSB mode was also investigated for the other types of OSBs. The results are shown in the figure 3.9. There are maximum dynamic displacement responses for each of the four different type OSBs at each site. The maximum displacements are plotted with OSB modes divided by the fundamental mode of the highway bridge.

Relatively large responses (see the box at top of Figure 3.9) occur at site 2 for all types of signs except the LED. The large displacements are a consequence of large volume of heavy trucks at this test site which produced larger bridge displacements and consequently larger OSB displacements. To remove effect of heavy traffic load , the mid-span displacements of highway bridges were computed by integrating the accelerations twice to provide an estimate of the bridge displacements. The resulting highway bridge displacements are given in Table 3.4. These displacement values were used to normalize the maximum displacements of OSBs and the result is presented in the Figure 3.10. The data from site 2 fall in with the data from the other sites when the displacement are normalized using the estimated mid span deflection.

Table 3.3 Natural Frequencies of OSB models

OSB span	Mode	No sign	Shape	Regular	Shape	FOS	Shape	LED	Shape
50'	1	3.14	T	2.53	T	1.94	T	2.03	T
	2	6.48	V+H	4.45	V+H	**3.98**	V+H	**3.89**	V+H
	3	6.64	V-H	6.14	V-H	4.72	V-H	4.71	V-H
75'	1	4.28	T	3.75	T	2.99	T	3.19	T
	2	5.91	V-H	4.17	V+H	**3.19**	V+H	**3.37**	V+H
	3	6.4	V+H	4.96	V-H	3.65	V-H	3.77	V-H
100'	1	2.86	T	2.6	T	2.2	T	2.27	T
	2	3.54	H	2.9	H+V	2.31	H+V	2.37	H+V
	3	**4.04**	V	**3.35**	V-H	2.48	V-H	2.61	V-H
120'	1	2.78	T	2.52	H+V	2.06	H+V	2.11	H+V
	2	2.88	H	2.6	T	2.22	V-H	2.32	-T+V
	3	**3.27**	V	**2.90**	V-H	2.27	T	2.33	V+T

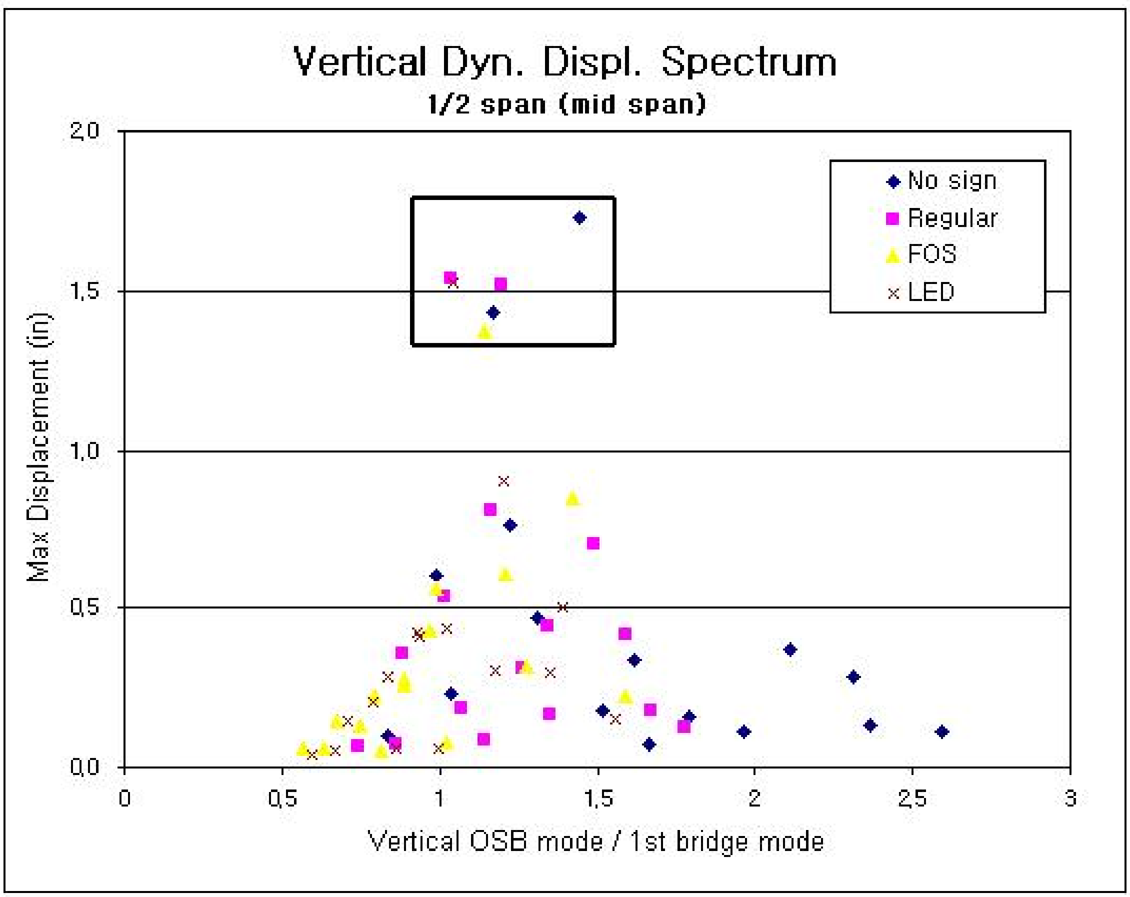


Figure 3.9 Vertical Dynamic Displacement Responses

Table 3.4 Maximum Mid-span Displacements of Highway Bridges

Field test	Site	Highway bridge span (ft)	Test number	Mid-span displ. of highway bridge (in)
1	Ben White 29	110	57	0.09
2	Ben White 30	120	18	0.16
3	Barton Creek (loop1)	130	13	0.06
4 & 5	US 183	106	32	0.06
			36	0.03

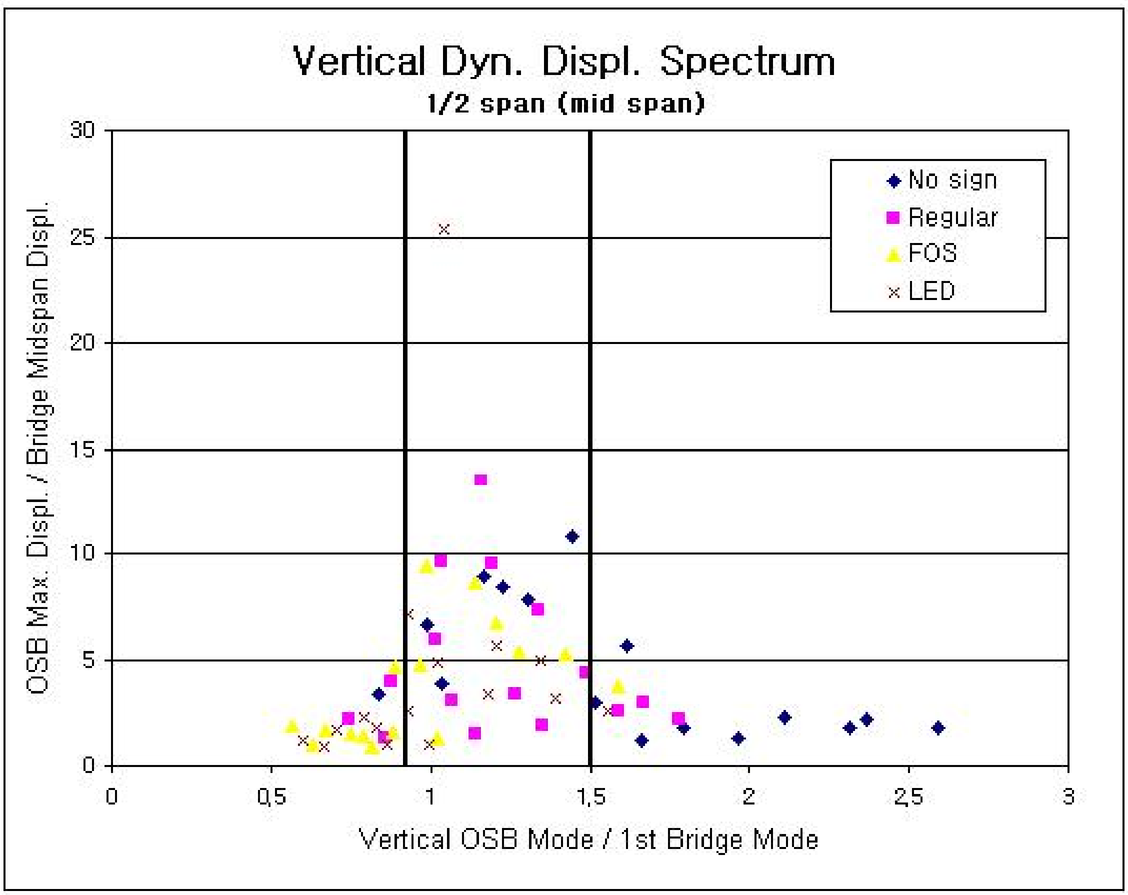


Figure 3.10 Normalized Vertical Dynamic Displacement Responses

The normalized vertical displacements of OSBs become very large when the ratio of the dominant modes of the OSB divided by the dominant frequency of the bridge is between 0.9 and 1.5. The range is shown by two vertical bars in the Figure 3.10. Horizontal displacement responses (appendix D) also have a similar range where they become very large. The displacements increase in the Figure 3.10 as frequency ratio approach 1.2. Table 3.5 was constructed by grouping the sign bridges spans with each type of sign into a frequency range. The darker shaded cells in the table are sign bridge spans and signs that are likely to undergo very large dynamic oscillations. The cells with no shading would have frequency ratios outside the bounds of 0.9 and 1.5 and should exhibit relatively small OSB vibrations. The lighter gray cells would have significant dynamic vibrations but less than those exhibited by structures defined by the darker cells.

It can be seen shorter span OSB's with either "FOS" and "LED" type signs are influenced much more by the vibrations of highway bridges then the longer sign bridges. The increased weight of the "FOS" and "LED" type signs lower the first mode frequency of the shorter span OSB's compared to that of bridge. The longer spans OSB's with the same signs have a lower frequency than the highway bridge and fall out of the range of where dynamic deflections would be cause for concern. OSB's with regular signs have a higher first mode which makes them most susceptible to dynamic oscillations when the OSB span is 75 to 100 feet. Longer and shorter spans are still in the range where large dynamic oscillation might occur. Replacing a regular sign with an FOS or LED on an existing 50 or 75 foot span OSB will result in an increase in dynamic oscillations particularly for 50 foot span OSB.

Table 3.5 Frequency Ranges of OSB Models

Sign type	Span length of OSB			
	50'	75'	100'	120'
No sign	6.5 ~ 6.6	5.9 ~6.4	3.5 ~ 4.0	2.9 ~ 3.3
Regular sign	4.5 ~ 6.1	4.2 ~ 5.0	2.9 ~ 3.4	2.5 ~ 2.9
FOS	4.0 ~ 4.7	3.2 ~ 3.7	2.3 ~ 2.5	2.1 ~ 2.2
LED	3.9 ~ 4.7	3.4 ~ 3.8	2.4 ~ 2.6	2.1 ~ 2.3

$$0.9 < \frac{\text{OSB mode}}{\text{Bridge mode}} < 1.5 \Rightarrow 2.3 < \text{OSB mode} < 5.9$$

$$1.0 < \frac{\text{OSB mode}}{\text{Bridge mode}} < 1.4 \Rightarrow 2.5 < \text{OSB mode} < 5.5$$

$$1.1 < \frac{\text{OSB mode}}{\text{Bridge mode}} < 1.3 \Rightarrow 2.8 < \text{OSB mode} < 5.1$$

Chapter 4 Conclusions and Recommendations

The following conclusions based upon the results of this study are:

1. The magnitude of bridge accelerations is proportional to the distance from the bridge pier to the location of the OSB. The maximum amplitude accelerations occurring at midspan. The peak accelerations at the 1/8 of the bridge span were 40% of the midspan accelerations.
2. The lowest two modes of the 106 to 130 foot span prestressed beam bridges included in this study were between 2.5 to 4.1 Hz.
3. The LED and FOS larger masses reduce the fundamental frequencies of the sign bridges relative to a standard highway sign and change the dynamic response of the sign bridges. Replacing a traditional sign with an LED or FOS on an OSB with a span of 75 feet or less will increase the vibrations of the sign bridge since it will lower the frequency of the OSB making it closer to the frequency of vibration of the bridge. Changing the type of sign on a longer span OSB, span greater than 100 ft., will lower the frequency of the OSB below the bridge frequency and will produce a reduction in vibration of the OSB.

The following recommendations are given based upon the results of this study:

1. Overhead sign bridges with spans less than 100 ft. should be mounted on pier caps and not within the spans of a highway bridge. If due to road geometry they must be mounted along the span, they should be mounted within 1/8 of the bridge span from the pier.
2. Changing a conventional sign to a FOS or LED sign lowers the natural frequency of the OSB and can result in a significant increase in the vibration in an OSB with spans of 75 ft. or less. It is recommended that an OSB mounted to the bridge not be changed to an FOS or LED sign, unless the OSB span exceeds 100 ft.
3. The walkway in front of a conventional sign should be removed if the OSB is mounted within the spans of a highway bridge. The removal of the cantilever walkway will significantly reduce the vibrations of the OSB. This is based upon the earlier study which investigated the dynamic behavior of OSB with and without the walkway and lights.

Appendix A

Member Size and Material Properties

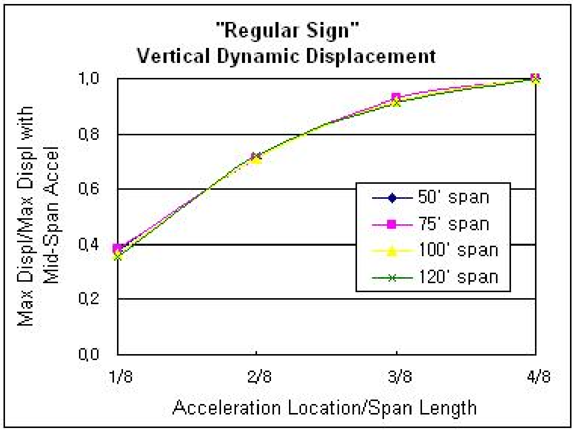
Base Truss Structure

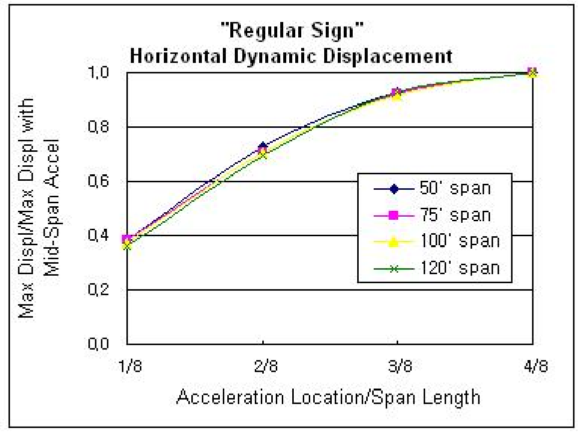
TYPE	COLUMN		
	COLUMN	BRACING	BEARING ANGLE
50' OSB	W10X22	2Ls 2.5 X 1.5 X 0.188	L 4 X 4 X 0.313
75' OSB	W16X36	2Ls 3 X 2.5 X 0.25	L 5 X 5 X 0.375
100' OSB	W14X30	2Ls 3 X 3 X 0.188	L 5 X 5 X 0.5
120' OSB	W14X34	2Ls 3 X 2.5 X 0.25	L 5 X 5 X 0.5

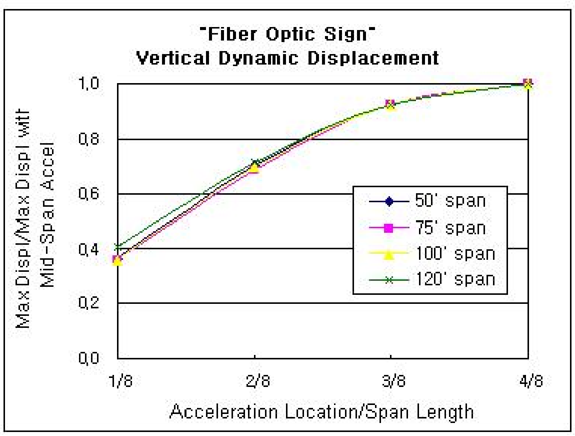
TYPE	HORIZONTAL TRUSS				
	CHORD	D-LOAD DIAGONAL	D-LOAD VERTICAL	W-LOAD DIAGONAL	W-LOAD STRUT
50' OSB	L 3 X 3 X 0.188	L 2.5 X 1.5 X 0.188	L 2.5 X 1.5 X 0.188	L 2.5 X 2.5 X 0.188	L 2 X 2 X 0.188
75' OSB	L 3.5 X 3.5 X 0.313	L 2.5 X 2.5 X 0.188	L 2.5 X 2.5 X 0.188	L 3 X 3 X 0.188	L 2 X 2 X0.188
100' OSB	L 3.5 X 3.5 X 0.375	L 3 X 2 X 0.188	L 3 X 2 X 0.188	L 3 X 2.5 X 0.25	L 2.5 X 2.5 X 0.188
120' OSB	L 4 X 4 X 0.438	L 3 X 2.5 X 0.188	L 3 X 2 X 0.188	L 3 X 3 X 0.25	L 2.5 X 2.5X 0.188

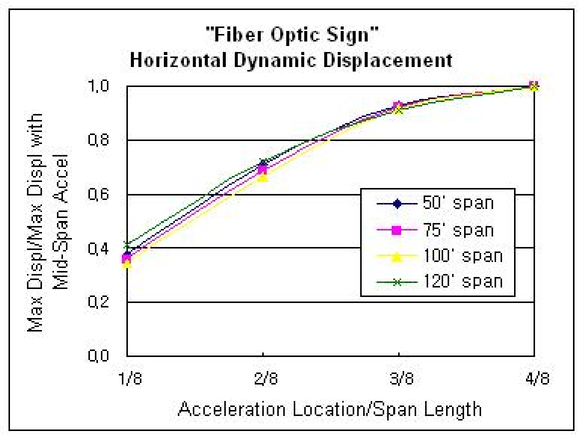
Appendix B

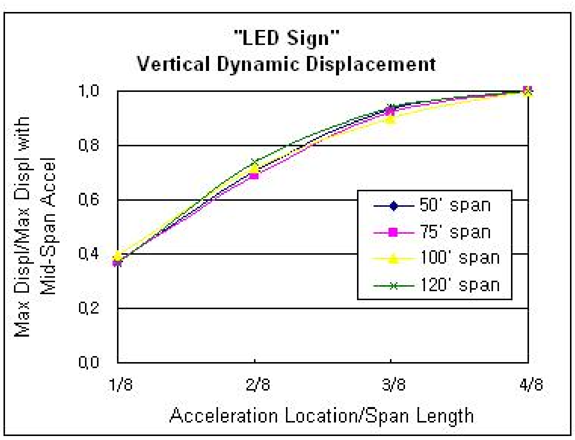
Dynamic Displacement Variation along Highway Bridge Span (Site 1)

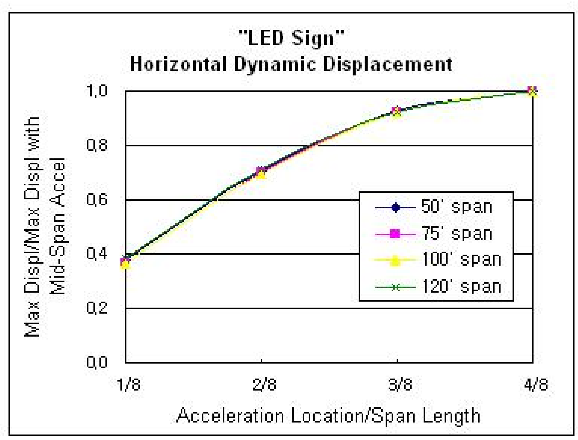








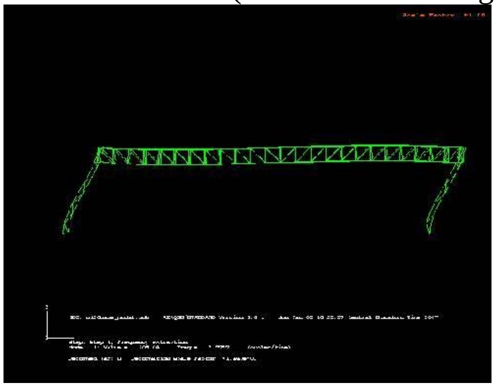


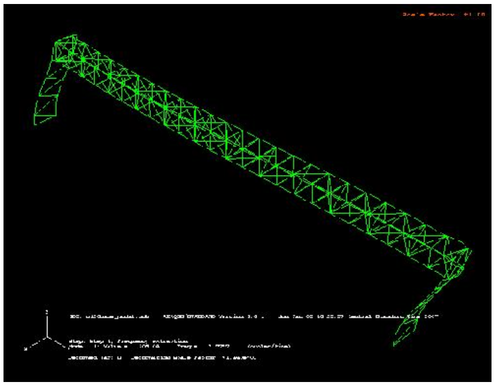


Appendix C

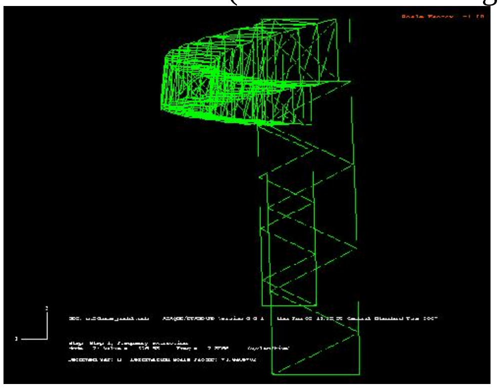
Mode Shapes of OSB Models

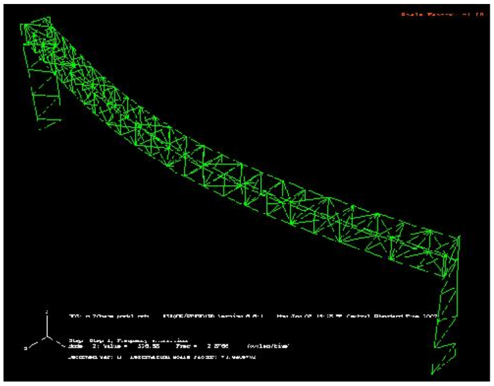
Transverse mode (1st mode of “No Sign”, 120’ span model)



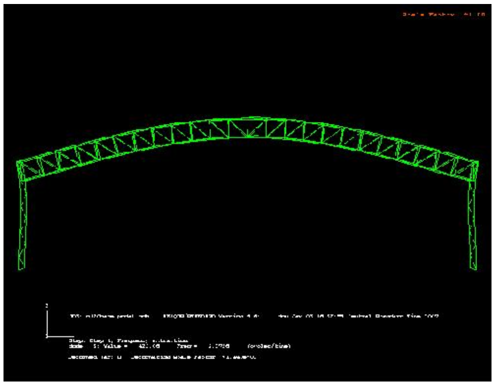


Horizontal mode (2nd mode of “No Sign”, 120’ span model)





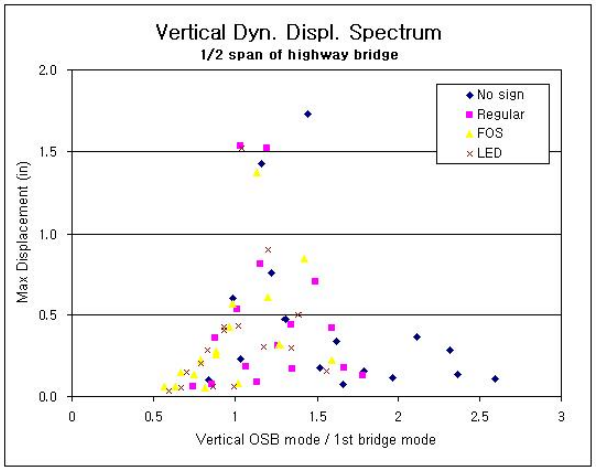
Vertical mode (3rd mode of “No Sign”, 120’ span model)

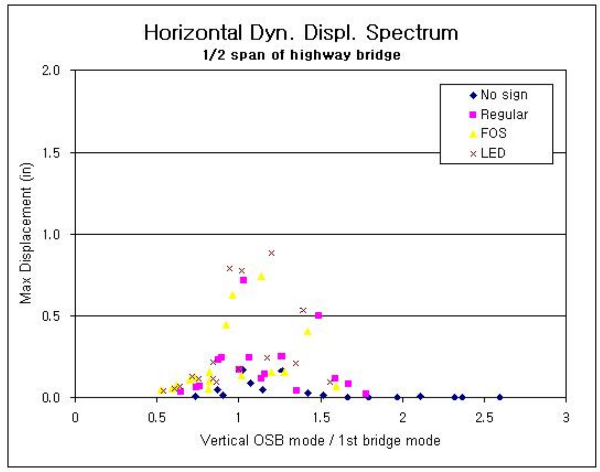


Appendix D

Dynamic Displacement Spectrum

Displacement Spectrum





Normalized Displacement Spectrum