SEISMIC LOAD DISTRIBUTION BETWEEN GIRDERS IN INTEGRAL BRIDGES

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ABSTRACT

This paper focuses on an investigation of the seismic lateral load distribution among girders in integral precast girder bridges using analytical and experimental results. For the analytical study, two approaches were investigated. First, an approach utilizing simple stiffness models to predict load distribution in integral bridge structures is presented. The second analysis approach utilized detailed grillage models, from which girder distribution values were extracted. The experimental data were gathered from several tests completed over the past 15 years, including two large scale tests completed by the authors. The experimental girder distribution values were based on strain or load cell data. In addition to comparing the analytical and experimental results, they will also be compared to AASHTO's current design recommendations. It is shown that the current design recommendations do not appropriately account for the amount of load that is distributed to the girders that are not adjacent to the column. Failing to appropriately distribute the load leads to excessive reinforcement, stiffness, and cost in the connection region of adjacent girders. Following an examination of the seismic lateral load distribution, suitable recommendations for more appropriate distribution of seismic lateral load in integral bridge superstructures is discussed.

Keywords: precast, load distribution, bridge, seismic, segmental, integral

INTRODUCTION

Integral bridge designs utilizing concrete columns provide several advantages over conventional designs, particularly for bridges in seismic regions. Primarily, a moment-resisting connection provides additional redundancy in resisting lateral earthquake loads. Also, an integral connection allows the development of a plastic hinge location in the concrete column just below the cap beam to provide additional energy dissipation during a seismic event. Further, integral connections provide increased headroom below the bridge along with a reduced overall bridge height. These benefits of integral bridges, along with many other advantages that are not mentioned here, have been well-documented in recent years[1,2].

Due to the many advantages, integral bridges have been used with increasing frequency in seismic regions over the past decade. However, design recommendations for such structures continue to be limited, particularly in the consideration of the distribution of lateral load among girders in the bridge superstructure. Most bridges in the U.S. are designed according to AASHTO standards[3,4]. However, AASHTO provides very little information on the distribution of lateral seismic loads. Bridge design in California, obviously a high seismic region, is primarily based on standards established by the California Department of Transportation (Caltrans). Caltrans' primary design recommendations related to seismic design can be found in *Caltrans Seismic Design Criteria*[5]. This document again provides very little information on lateral load distribution. Caltrans' *Bridge Design Aids*[6] also contains general bridge recommendations, but this resource too provides little insight related to lateral load distribution. A brief investigation of other general bridge recommendations from other state departments of transportation that have done considerable work related to seismic design of bridges reveals nothing significant on topics related to lateral load distribution[7,8].

Despite the lack of design recommendations on the issue of lateral seismic load distribution, several investigations in the last decade or so that were intended primarily to examine the performance and sufficiency of bridge structures for large-magnitude earthquake events have investigated load distribution as a secondary issue. One of these projects, conducted by Holombo et. al.[9] at the University of California-San Diego in the late 1990s investigated a 4-girder prestressed concrete superstructure. NCHRP Project 12-54[10], conducted jointly by Iowa State University and Modjeski and Masters, Inc., from 2000 to 2003, investigated two similar bridges with 4-girder steel superstructures. A more recent investigation under the guidance of Caltrans examined and tested a 5-girder prestressed concrete superstructure[12].

All of these tests had specific areas of focus; however, common areas of interest for each of these tests can be summarized as: (1) the design of a prototype bridge utilizing integral connection details capable of withstanding seismic load, (2) experimental validation of these details using large-scale tests, and (3) the formation of suitable design recommendations and specifications based on the analytical and experimental findings.

This paper compiles the results related specifically to lateral load distribution from these experimental tests and compares them with lateral load distribution predictions from simple stiffness and grillage models for each of the structures. The seismic lateral load distribution in each test exhibited considerable variation from current design recommendations. More suitable recommendations for the distribution of the lateral load would improve seismic design practice for integral bridges.

CURRENT DESIGN APPROACH

The primary guidelines for bridge design in the United States originate from AASHTO. While the AASHTO specifications devote a considerable amount of information to the distribution of vertical loads for various types of girder and superstructure configurations, considerably less attention is given to the distribution of lateral loads. Section 4.11.2 in the recently released *AASHTO Guide Specifications for LRFD Seismic Bridge Design*[4] states that superstructure components and their connections "shall be designed to resist overstrength moments and shears of ductile columns." Section 8.10 in these guidelines goes on to address the capacity design of the superstructure for integral bent caps in reinforced concrete structures. These guidelines limit the distribution of the column overstrength moment to an effective width equal to the sum of the diameter of the column and the depth of the superstructure. The practical conclusion of this limited effective width is that the lateral load can rarely, if ever, be distributed to the exterior girders, making the bridge design less cost effective. If the AAHSTO guidelines are followed for the four prototype structures noted above, the distribution of any portion of the column overstrength moment to the exterior girders is not allowed in any of the structures. The consequence of current guidelines and practice is that the girder(s) immediately adjacent to the column are required to be designed to carry the entire overstrength moment, while experimental work has in actuality shown that much of the moment from the column is distributed through the integral cap beam to non-adjacent girders. The current design approach does not adequately consider the true behavior of integral bridge superstructures.

Bridge design by Caltrans typically represents state-of-the-art approaches to seismic-related issues. The primary bridge guidelines currently implemented by Caltrans are the *Bridge Design Aids* (BDA)[6] and the *Seismic Design Criteria* (SDC)[5]. Chapter 5 of the BDA offers limited information on girder design and load analysis, but there is no information in the BDA provided on lateral load distribution. Section 7.2 in the SDC follows the same definition for effective width in resistance to column overstrength moment as the AASHTO recommendations cited in the above paragraph, which results in no distribution to the exterior girders of the structures considered in this investigation. This section of SDC does, however, add the recommendation that "the effective superstructure width can be increased at a 45^{o} angle as you move away from the bent cap until the full section becomes effective."[5] Thus, for the four prototype structures being considered here, distribution of the lateral load to the exterior girders would not be allowed at the cap-to-girder connection, but it would be allowed in regions further from the cap beam.

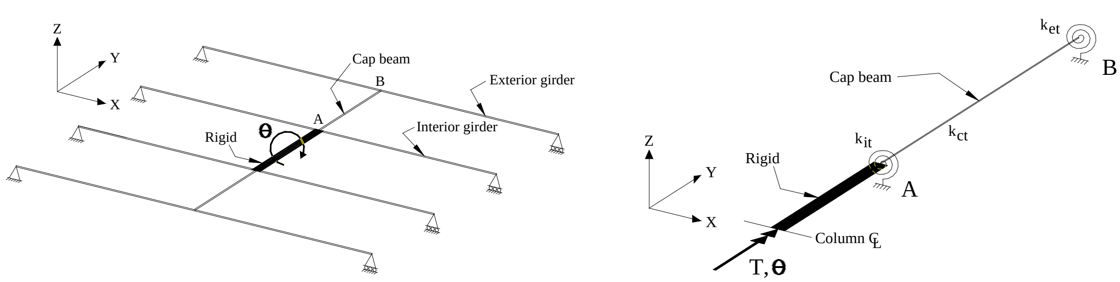
ANALYTICAL APPROACHES

Common approaches for the analysis of integral bridge structures for seismic regions have included using finite element analysis (FEA) and grillage model analysis. While both of these approaches can be useful and produce reasonable results, they can be cumbersome and time-consuming from a design perspective. In an attempt to simplify the process of predicting load distribution for integral bridges, analytical models that are stiffness-based are an alternative for approximating the distribution of seismic loads through the girders of an integral bridge superstructure. These models are far less complex than typical analytical grillage and finite-element models; yet the use of them in this investigation has shown their results to be reasonably consistent with the experimental results. The following sections present more detail in the development of the analytical models that have been used in this study in the investigation of seismic load distribution.

SMST MODELS FOR LATERAL LOAD

As mentioned, simple stiffness (SMST) models have been found to be useful in providing a simpler alternative to grillage models or FEA models that are commonly used in analytical predictions and investigations of the seismic behavior of structures. The SMST model is developed to determine the distribution of the seismic moment developed at the top of the column to the girders in the superstructure. A schematic of the horizontal load distribution concept is shown in Figure 1a. The symmetrical nature of most bridge superstructures typically allows the elimination of half of the superstructure in developing the stiffness model, as shown in Figure 1b. By using the appropriate relationship between the torsional stiffness of the cap beam, k_{ct}, and the flexural stiffnesses of the interior and exterior girders, k_{it} and k_{et}, the reactions at A and B can again be easily calculated given either the moment, M, or the rotation, θ, at the column-to-cap beam connection.

For the lateral load case, the column overstrength moment produces a torsional load in the cap beam. Thus, the member stiffness of the cap that is used in the model is the torsional stiffness, while the member stiffnesses in the girders that are used are the flexural stiffness



(a) Torsional load on cap beam due to column lateral load

(b) Cap beam region with girder stiffnesses identified as springs

Fig. 1 Lateral Load Distribution Schematics values. Also, note that torsion in the cap results in girder curvature that is downward on one side of the cap beam and upward on the other side; thus, the flexural stiffness values in the girder have to be adjusted accordingly since the composite girder-and-deck section properties are not typically symmetrical about the horizontal axis.

An appropriate combined stiffness relationship can be developed by observing that, for a given configuration, the contribution of individual member stiffness values will contribute towards the resistance of a load in a manner that is either simultaneously in parallel or sequentially in series with other individual member stiffness components. Observing this relationship and developing it mathematically, the resulting total stiffness value, k_{lat}, for the typical lateral load configuration shown in Figure 1b is:

$$k_{lat} = \frac{k_{et} k_{ct}}{k_{et} + k_{ct}} + k_{it} \qquad (1)$$

For further details on the development of the analytical model for horizontal load distribution, refer to Staudt [11].

The accuracy of this approach is dictated in large part by the appropriateness of the member stiffness values that are used. Much work has been done with regards to appropriate section properties to use for reinforced concrete sections, and some of this work has been devoted specifically to the behavior of reinforced concrete under seismic loading. A particularly useful reference in regards to appropriate section properties for concrete sections is Priestley, et. al.[13]

GRILLAGE MODELS

For each of the four superstructures considered, grillage models or slight variations were developed and used for comparison with the SMST model and the experimental results. Member sections that were used in the grillage analyses included the longitudinal composite sections of the girders and bridge deck, the transverse bridge deck sections, transverse diaphragm sections, the cap beam sections, and the concrete column sections. A combination of uncracked and cracked concrete deck properties were used in the composite girder sections, depending on the girder moment magnitude and direction. An additional nonlinear component of the grillage models used in these investigations is the nonlinear spring located in the plastic hinge region of the reinforced concrete column. The spring properties consisted of appropriate moment-curvature relationships for the plastic hinge determined by appropriate analytical methods. For additional information on column plastic hinge moment-curvature behavior, see Priestley et. al.[13], and more detailed information on the development of the grillage models can be found in Vander Werff [1], Snyder [2], and Holombo et. al.[9]

It is worth noting that in the grillage analyses investigated in this paper, the behavior of the deck and diaphragm members connecting the girders in the transverse direction was at times observed to play a noticeable role in the load distribution among girders. In particular, the deck contribution has been observed to redistribute the seismic moments between girders when the critically loaded girders experience degradation in the connections and lower the flexural stiffness of these girders. It should be realized that the nature of stiffness-based SMST models and the grillage models as routinely developed may not produce accurate results when the contribution of the deck and diaphragm is not included.

FINITE ELEMENT MODELS

The deck and diaphragm behavior mentioned in the previous paragraph can perhaps be modeled even more thoroughly using a complete finite element analysis. While finite element results were not included as a particular component of this load distribution investigation, two of the projects referenced here did use finite element analyses in addition to grillage models. Again it was observed that, although there were some differences in the analytical results from the FEA models and grillage models, both techniques predicted significant distribution of the lateral load beyond the girders immediately adjacent to the column. For further detailed information on the FEA work on one of the investigations presented below, refer to Theimann[14].

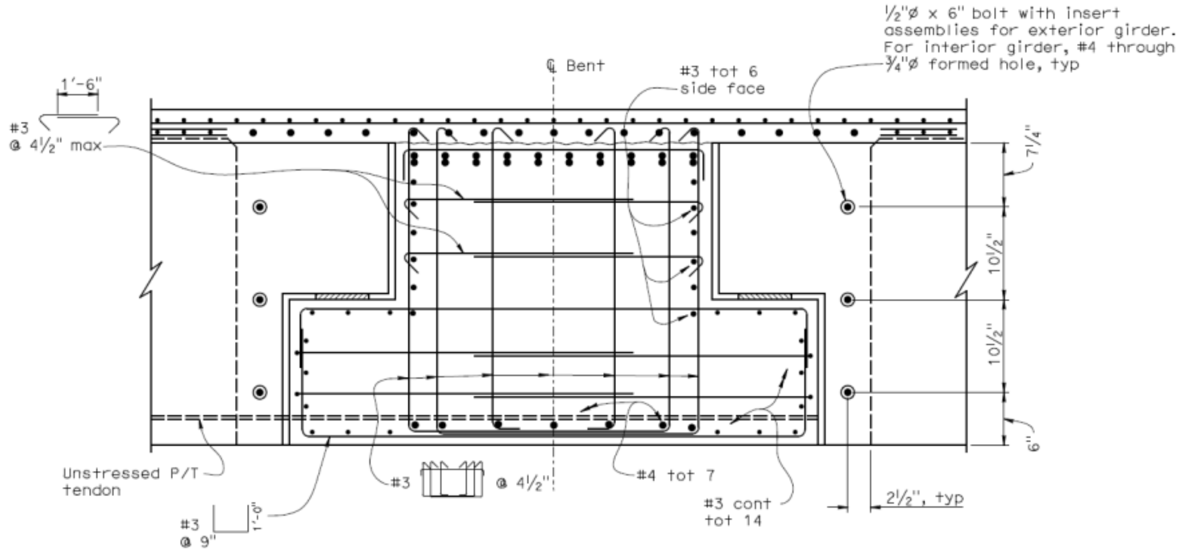
SUMMARY OF LARGE-SCALE TESTS

Four different prototype structures were considered as a part of this load distribution study. All of these structures have been modeled experimentally with large-scale test units intended to examine and quantify system performance. All of the prototype structures considered were integral bent cap, single-column structures, utilizing single reinforced concrete columns.

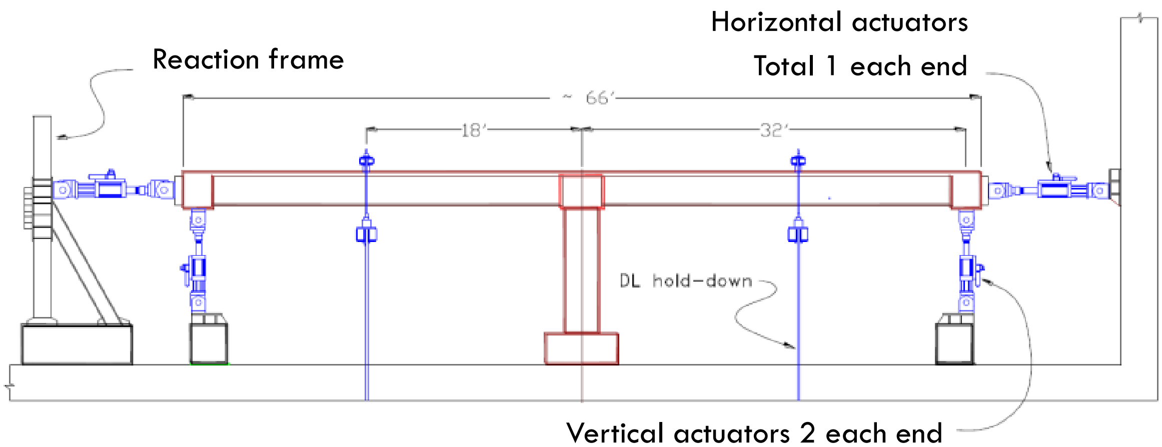
For each of the experimental studies that have been considered in this investigation, detailed finite element and/or grillage models were developed to predict their load behavior. In each test, the analytical models proved to predict reasonably well the behavior of the experimental specimens. While the system behavior has been well-documented for each of these tests, careful investigation of the load distribution among the girders has never been completed. It is interesting to note that, for each of these tests, the lateral load distribution is fairly consistent, both in the analytical estimates as well as the experimental results. When comparing the lateral load distributions to current design recommendations, the results from all of these tests consistently reveal that there is far more distribution of lateral load among the girders than what is specified in current design recommendations. The following brief sections highlight the experimental results from each of the experimental studies considered.

5GC1: CONCRETE CAP BEAM WITH FIVE PRECAST GIRDERS

The test unit for structure 5GC1 is shown in Figure 2. The half-scale test unit modeled a portion of the reinforced concrete column, the cast-in-place concrete cap beam, and the central portion of the five precast concrete I-shaped girders on both sides of the cap beam. Two different integral connection details between the girders and cap beam were utilized, one on one side of the cap beam and the other on the opposite side. The first connection detail implemented was a detail that has already been used by Caltrans, generally referred to



(a) Girder-to-cap connection



(b) Test configuration

Fig. 2 Test Unit 5GC1

as the as-built connection of this test unit, and shown on the right side of Figure 2a. The connection detail used on other side of the cap beam was similar but utilized an unstressed post-tensioning strand running through the bottom flange of the girder, into the cap beam, and to a termination on the far side of the cap beam. This connection, referred to as the improved connection, is shown on the left side of Figure 2a, with the termination ending on the right side of the cap beam as shown in the figure. For purposes of simplicity, and based on post-test data that was available due to the experimental instrumentation configuration, the data used in the distribution analysis presented in this paper are from only the improved connection side of the cap beam. Some brief comparative work between the two connection types has indicated that, with regards to load distribution, both connection types produced similar behavior.

Figure 3 provides the force-displacement hysteresis for the test unit when subjected to simulated horizontal seismic loading. The system was observed to perform very well. The superstructure provided sufficient strength to successfully form a plastic hinge at the column top and bottom ends, and the structure maintained strength up to displacement ductility 8 with only minor strength loss at ductility 10. More information on both connection types and the experimental results can be found in Snyder et. al.[12]

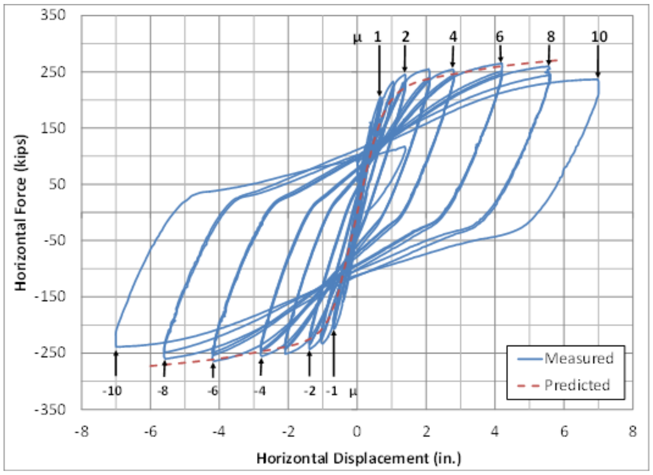


Fig. 3 Load-Displacement Response for 5GC1

4GS1 AND 4GS2: STEEL BOX CAP BEAM WITH FOUR STEEL GIRDERS

The prototype superstructures for structures 4GS1 and 4GS2 consisted of a steel box beam pier cap integrally connected to four steel girders. The test unit for 4GS1 utilized a deeper superstructure, while the weight of the superstructure was reduced in 4GS2. The test units were configured similarly to 5GC1 shown above, but they were constructed in an inverted configuration to simplify the laboratory setup and load simulation, with dead load appropriately modeled by use of a vertical actuator. Both 4GS1 and 4GS2 exhibited elastic response throughout the duration of the test, and a plastic hinge was successfully formed in the column of each test unit. More detail on the design and experimental work related to structures 4GS1 and 4GS2 can be found in Vander Werff [1], Wassef et. al.[10], and Sritharan et. al.[15]

4GC1: CONCRETE CAP BEAM WITH FOUR PRECAST GIRDERS

The prototype superstructure for structure 4GC1 consisted of a post-tensioned concrete cap beam and four precast, prestressed concrete bulb-tee girders. The test unit was constructed in a similar fashion as 5GC1 presented earlier. The test unit was observed to exhibit very good seismic behavior, retaining strength all the way to displacement ductility 8. More information on work related to structure 4GC1 can be found in Holombo et. al.[9]

COMPARISON OF ANALYTICAL AND EXPERIMENTAL LOAD DISTRIBUTIONS

SMST models of each of the four structures were developed to investigate the distribution of the moment in the girders due to the lateral load. Results from grillage model analyses of each of the test units were used to look at the distribution of lateral load moment. Experimental results from each of the test units were then compared to the analytical predictions along with the current design recommendations for lateral load distribution. Since none of the test units were subjected to horizontal-load-only conditions, the experimental results from the horizontal load tests for each of the structures were biased by removing the vertical load contribution from the strain or load data that was recorded. This biasing was accomplished by carefully looking at the zero-horizontal-load conditions during each cycle of the horizontal load tests and investigating the consistency of the vertical distributions at the instances where there was no horizontal load present.

Table 1 lists the results from the lateral load experimental investigations, design recommendations, and analytical predictions. A few clarifications on the development of this table follow. First, the ratios for the experimental and analytical distributions that are reported in this table were determined on the basis of total load in all girders. Hence, for a five-girder structure, if the load was evenly distributed among all five girders, the resulting ratio would be 0.20 for the center girder, 0.20 for the intermediate girder, and 0.20 for the exterior girder, since there were two intermediate and two exterior girders. Likewise, for a four-girder structure, the distribution would be 0.25 for the interior girder and 0.25 for the exterior girder, since there were again two of each girder. The second clarification is that the "difference" listed for each model is the percentage difference between the analytical prediction and the experimental result. Thus, the "Design Difference" of 62.4% recorded in the top row of the table indicates that the ratio predicted by current design recommendations was 62.4% higher than the ratio that was determined experimentally. Likewise, the "Grillage Difference" of 2.3% indicates that the ratio predicted by the grillage model was 2.3% higher than the ratio that was determined experimentally. Note that the design ratios reported in this table were determined using AASHTO and current design recommendation approaches as described previously in this paper.

The first observation regarding the numbers reported in Table 1 is the striking dissimilarity of the design ratio numbers to the actual experimental numbers. Current design recommendations allow lateral load distribution among only the center and intermediate girders of structure 5GC1 (the only 5-girder structure included in this investigation). However, the experimental results revealed that 15.8% of the lateral load was carried by each of the exterior girders, meaning that the two exterior girders together carried almost 32% of the total lateral load as opposed to the design assumption of zero percent. The 4-girder structures (4GS1, 4GS2, and 4GC1) revealed even less correlation with current design recommendations. For these structures, current design guidelines again allow no distribution of lateral load to the exterior girders; however, the experimental results revealed that 34%, 30%, and 33% of the total lateral load was distributed to the exterior girders in structures 4GS1, 4GS2, and 4GC1, respectively. These results indicate that current design recommendations are overly conservative in containing the lateral load in the girders adjacent to the columns.

While there is some discrepancy in the analytical predictions for each of the four structures considered, the analytical results for all of the structures were closer to the experimental results than the design recommendations for all of the structures. Looking at the grillage analyses, the maximum percentage difference between the predicted ratios and the experimental ratios was 18.2%. All of the grillage models predicted significant distribution of the seismic load to the exterior girders, and the experimental results verified these predictions.

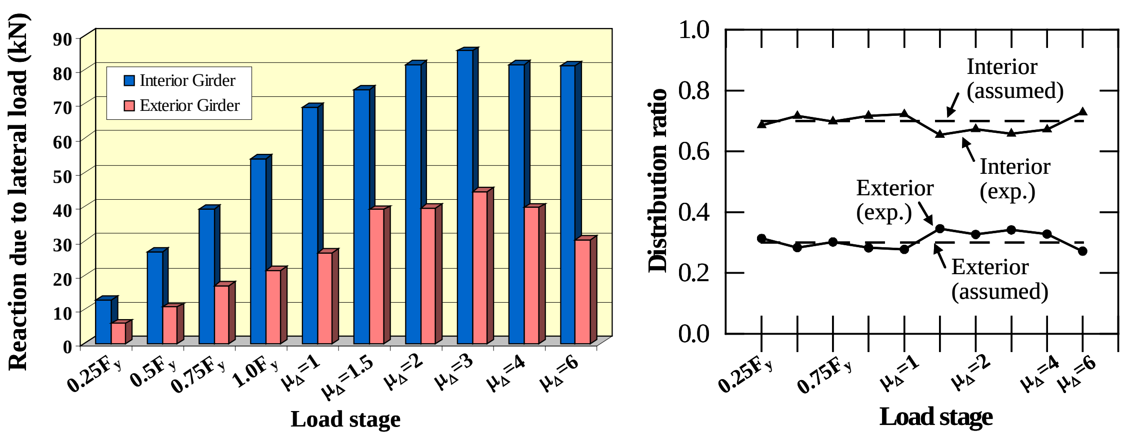
The SMST models also seem to provide much better comparisons to the experimental results than current design predictions. The largest discrepancies in the SMST predictions are for 5GC1 (the 5-girder structure), where the SMST model predicts more load in the center girder, when in fact the experimental results revealed more load being transferred to the intermediate girders. This discrepancy appears to be related to the inherent form of the SMST model, which is stiffness-based, since the center girder is immediately adjacent to the column and thus behaves as a relatively short and stiff element in carrying load away from the rest of the structure. The grillage and experimental results both revealed that, in actuality, a large portion of the load was distributed away from the center girder and into the remainder of the structure. The other SMST models provided good predictions, with a maximum ratio discrepancy of about 0.04. In fact, the variation of even the 5-girder structure was less than 0.08. Thus, the experimental results validated the predictions of both the grillage analyses and the SMST analysis, confirming that large portions of the lateral load are indeed distributed beyond the girders that are immediately adjacent to the column.

Table 1: Lateral load distribution comparison

Test Unit 5GC1							
	Experimental Ratio	Design Ratio	Design Difference	Grillage Ratio	Grillage Difference	SMST Ratio	SMST Difference
Center	0.205	0.333	62.4%	0.228	2.3%	0.281	37.1%
Intermediate	0.239	0.333	39.3%	0.212	-11.3%	0.198	-17.2%
Exterior	0.158	0	-100.0%	0.174	10.1%	0.161	1.9%
Test Unit 4GS1							
	Experimental Ratio	Design Ratio	Design Difference	Grillage Ratio	Grillage Difference	SMST Ratio	SMST Difference
Interior	0.33	0.5	52.1%	0.36	9.5%	0.369	12.3%
Exterior	0.17	0	-100.0%	0.14	-18.2%	0.131	-23.7%
Test Unit 4GS2							
	Experimental Ratio	Design Ratio	Design Difference	Grillage Ratio	Grillage Difference	SMST Ratio	SMST Difference
Interior	0.35	0.5	42.9%	0.349	-0.3%	0.353	0.7%
Exterior	0.15	0	-100.0%	0.151	0.7%	0.147	-1.7%
Test Unit 4GC1							
	Experimental Ratio(4)	Design Ratio	Design Difference	Grillage Ratio(3)	Grillage Difference	SMST Ratio	SMST Difference
Interior	0.3335	0.5	49.9%	0.344	3.1%	0.305	-8.5%
Exterior	0.1665	0	-100.0%	0.156	-6.3%	0.195	17.1%

LATERAL LOAD DISTRIBUTION AT VARIOUS LOAD LEVELS

One of the questions regarding lateral load distribution in superstructure girders is whether the distribution varies from low load levels to higher load levels. Test Units 4GS2 and 5GC1 provided useful data for investigating this issue. Figure 4 shows the experimental load distribution for 4GS2 for the peak conditions throughout the test, beginning at service load levels and continuing through several cycles of plastic deformation. It is seen here that the girder load distribution to the exterior girders seems to have begun almost immediately, at the first recorded load level. Already at a load level of only 0.25 F_y, where F_y defines the yield strength of the test unit, the exterior girders were carrying approximately 30 percent of the lateral load. Thus, it follows that formation of extensive deck cracking and high seismic loads are not required to engage all girders in resisting the column seismic moment in bridges with multiple girders and no soffit slab. Another observation from these results is that, not only is there significant distribution of load to the exterior girders at low load levels, but also the distribution to the exterior girders remains adequately consistent throughout the duration of the load test. Thus, such distribution can be relied upon for the bridge to resist loads at the service level as well as at higher load levels.

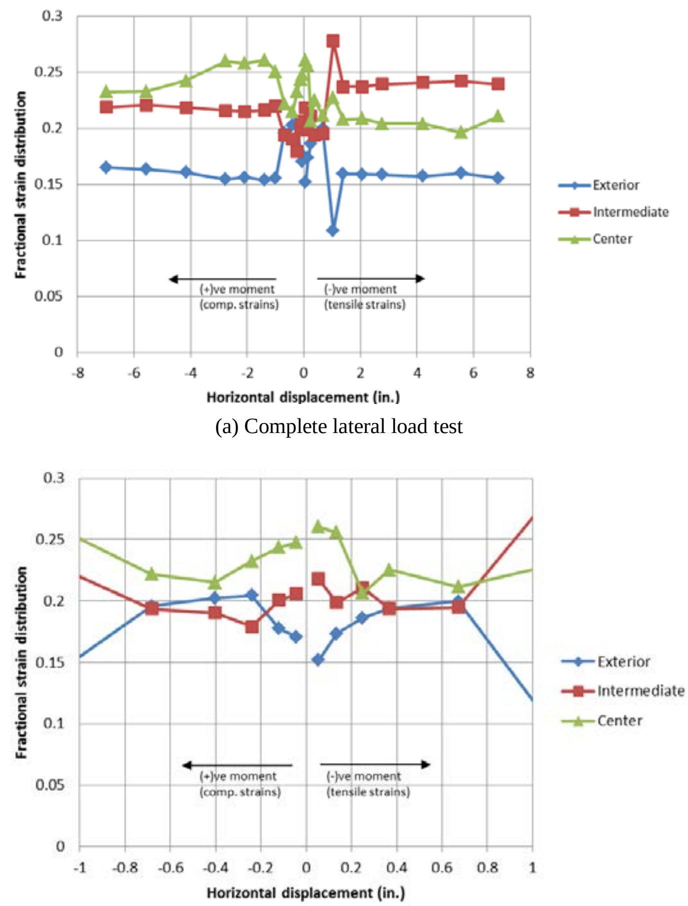


(a) Reaction obtained from load-cell data (1 kN = 0.225 kips)

(b) Lateral load distribution factor based on data in (a)

Fig. 4 Experimental Lateral Load Distribution for 4GS2

Figure 5 uses a slightly different form to show the load distributions at various load levels for test unit 5GC1. Figure 5a shows the distribution among the exterior, intermediate, and center girders for all of the peak conditions. Figure 5b shows the same data for the peaks of only the lower load conditions. Looking at these low-load distribution results, it is seen that, similar to 4GS2 in Figure 4, significant distribution to all the girders was measured already at the lowest load levels. The exterior girders are seen, at the very first peak recorded, to individually carry 15 percent (meaning that combined they are carrying 30 percent) of the total lateral load. These results from test unit 5GC1 concur with test unit 4GS2, revealing significant lateral load distribution is occurring already at low-level loads. Although there is a bit of irregularity in the distribution for the next four peaks recorded, as should be expected due to progressive cracking of the superstructure, significant distribution is observed even through this irregular pattern, and more uniform distribution is documented for all of the higher peak conditions. Thus, these results from test unit 5GC1 also confirm what test unit 4GS2 exhibited, namely, that the lateral load distribution continues through high ductility levels due to large lateral loads.



(a) Complete lateral load test

(b) Lateral load test, small loads only

Fig. 5 Lateral Load Distribution for 5GC1

CONCLUSIONS

This paper investigated the seismic lateral load distribution between girders in integral bridges using experimental data and analytical results from four test units. Based on this investigation, the following conclusions regarding the distribution of seismic lateral load in bridge superstructures can be made.

1. Current practice and recommendations related to seismic lateral load distribution for integral bridge structures are too conservative in the amount of lateral load distributed beyond the girders that are immediately adjacent to the column. All of the experimental investigations considered in this study showed as much as 30% of the lateral load being distributed all the way to the exterior girders. While the grillage model and simple stiffness model predictions produced comparable load distribution, current design recommendations assume no transfer of loads to exterior girders.
2. The lateral load distribution was found to be fairly independent of the lateral load level, as the distribution was observed to occur already at very low levels of lateral load.

3. The grillage model analyses were seen to predict the load distribution satisfactorily, and the analyses based on simple stiffness models were found to match reasonably well with the more complex grillage analysis techniques.
4. Based on the experimental and analytical results, it appears that 24% of the total lateral load may be assumed to transfer to the exterior girders of a four-girder structure. The resulting design ratios for this recommendation, in a form similar to that presented in Table 1, are 0.38 and 0.12 for the interior and exterior girder, respectively.
5. Based on the experimental and analytical results, a conservative design assumption of 40% of the total lateral load to the intermediate girders and 20% to the exterior girders of a five-girder structure may be appropriate. The resulting design ratios are 0.4, 0.2, and 0.1 for the center, intermediate, and exterior girder, respectively.
6. The ability to distribute the lateral load beyond immediately-adjacent girders will allow for more economical design of the superstructure, because current design practice conservatively underestimates the amount of load that is distributed laterally from the center of the superstructure to the more extreme girders.

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