Development of an Improved Protocol for Structural Evaluation of Dowel Load Transfer Systems for Concrete Pavements

Final Report
August 2021

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16. Abstract

This report describes the development of a test protocol for evaluating the structural behavior of alternate dowel load transfer systems for concrete highway pavement. The conceptual development and physical verification of the test are described, along with recommendations for incorporating the test in specifications for highway paving dowels. The test is based on the load-deflection test that is included in Section 5 of AASHTO T 253-02, a double-shear test of single dowels, except that the new test uses a circular load at the slab corner (rather than a 4,000 lb distributed load) and is applied to dowel groups across 4 ft wide concrete pavement joints (simulating a wheel path) rather than to single dowels that cross 1 ft wide joints. In addition, the test includes provisions for an optional 50% overload test condition.

Replicate tests were performed using standard 1.25 in. epoxy-coated steel dowels in the AASHTO T 253 test and the new test. Finite element data and test results were analyzed to develop recommended evaluation criteria for the new test that would be consistent with the original AASHTO T 253 test but applicable to multidowel systems rather than individual dowels. Adoption of this new test protocol will allow agencies to more easily evaluate and implement newer, more efficient, and more durable pavement dowel technologies with confidence.

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DEVELOPMENT OF AN IMPROVED PROTOCOL FOR STRUCTURAL EVALUATION OF DOWEL LOAD TRANSFER SYSTEMS FOR CONCRETE PAVEMENTS

Final Report
August 2021

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INTRODUCTION

The American Association of State Highway and Transportation Officials (AASHTO) approved AASHTO M 254, Standard Specification for Corrosion-Resistant Coated Dowel Bars, and AASHTO T 253, Standard Method of Test for Coated Dowel Bars, around 1975 when concrete pavement dowels were almost exclusively cylindrical steel bars, often coated with epoxy, paint, or other similar corrosion barriers. The AASHTO standards were developed to be directly applicable to epoxy-coated cylindrical steel dowels, and the included structural tests and acceptance criteria were developed around the behavior of single 1.25 in. diameter epoxy-coated cylindrical steel dowels (see Figure 1).

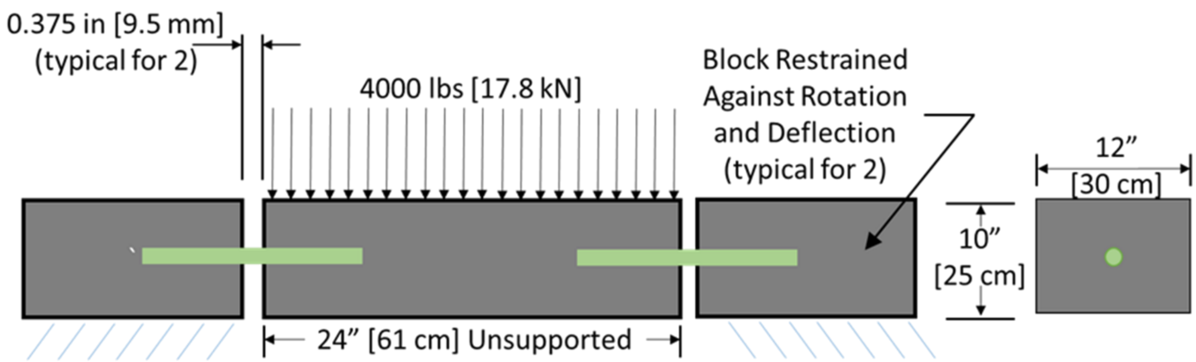

Figure 1. Schematic of AASHTO T 253 load-deflection test for concrete pavement dowels, with relative deflection limited to 10 mils (0.01 in.) [0.25 mm]

Innovations in dowel design, structural materials, and coating materials have resulted in the development and deployment of alternative dowel products, including the following:

- Plate dowels
- Hollow/tubular dowels (with or without coatings)
- Elliptical dowels (various materials)
- Fiber-reinforced polymer/plastic (FRP) dowels
- Composite dowels (FRP/steel, zinc/steel, etc.)
- Through-alloy dowels (stainless steel, microcomposite alloys)

Many of these newer products have different structural behavior than conventional 1.25 in. diameter solid steel dowels. Some manufacturers recommend the use of alternate dowel sizes or spacing with their products to produce joint behavior similar to that of 1.25 in. diameter steel dowels. Even conventional cylindrical steel dowels are sometimes used with nonuniform spacing for more efficient use of materials and are commonly used in sizes other than 1.25 in. diameter.

Problem Statement

Current AASHTO T 253 structural tests of single dowels are incapable of providing the information necessary to effectively assess the potential behavior of many newer dowel product systems (i.e., groups of dowels), which inhibits concrete pavement innovation and performance improvements. A new test is needed to better assess the potential of pavement dowel systems to provide adequate joint load transfer and joint stability.

Objectives

The objectives of this research were as follows:

- Develop and validate a load-deflection test procedure based on the AASHTO T 253-02 Section 5 procedure that could be used to assess the structural behavior of groups of pavement dowels.
- Characterize the relationship between the original AASHTO T 253 test results and the new test results so that a relative deflection limit can be determined for the modified test that represents structural equivalence to the deflection limit set forth for the original test.

The successful adoption and implementation of the new load-deflection test will allow agencies to objectively assess the structural behavior of newer, innovative dowel products and facilitate their adoption, resulting in the potential for more cost-effective construction and improved pavement performance. It will also provide dowel manufacturers with evaluation criteria for consideration in the development of improved and optimized dowel load transfer systems.

RESEARCH APPROACH

In early 2016, concepts for a modified version of the AASHTO T 253 load-deflection test were presented to the National Concrete Consortium. The new test carried forward the double-shear test concept of the original AASHTO T 253 load-deflection test but expanded the test specimen to a 4 ft width to simulate a pavement wheel path and accommodate a group of dowels. Additionally, the load configuration was changed from a 4,000 lb distributed line load to a 9,000 lb load applied to a 12 in. diameter circular plate system, similar to a falling weight deflectometer test and simulating the static application of one half of an 18,000 lb single-axle load (single wheel).

This test concept was selected for further development and validation. Schematics for the test are presented in Figures 2, 3 and 4.

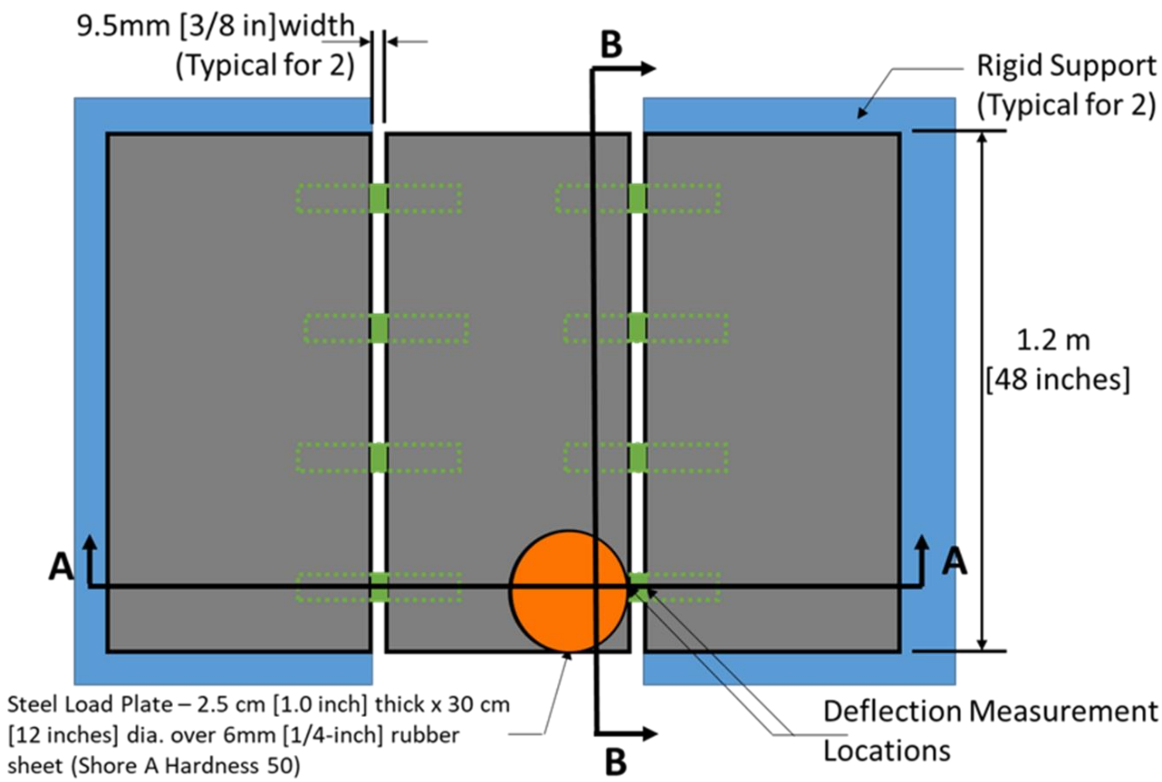

Figure 2. Proposed modified AASHTO T 253 dowel load-deflection test – plan view

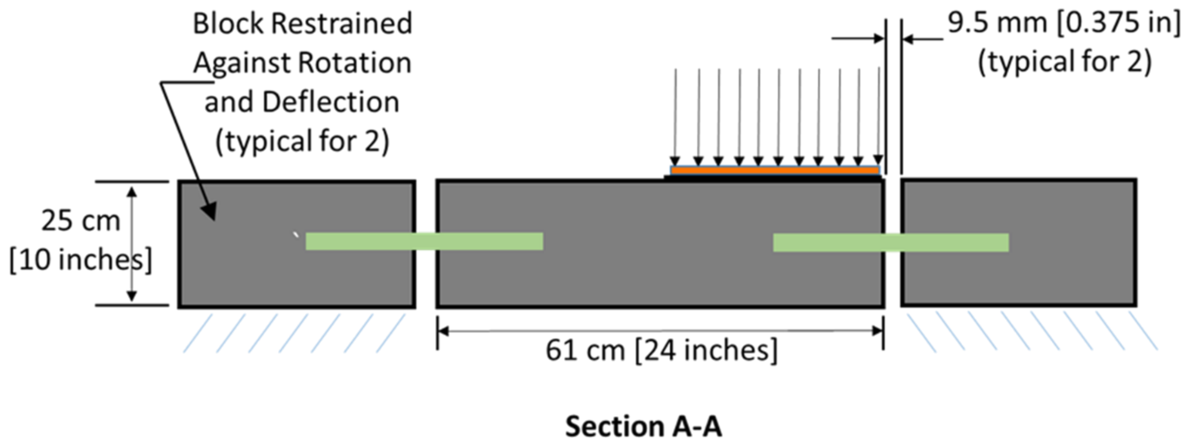

Figure 3. Proposed modified AASHTO T 253 dowel load-deflection test – elevation section A-A

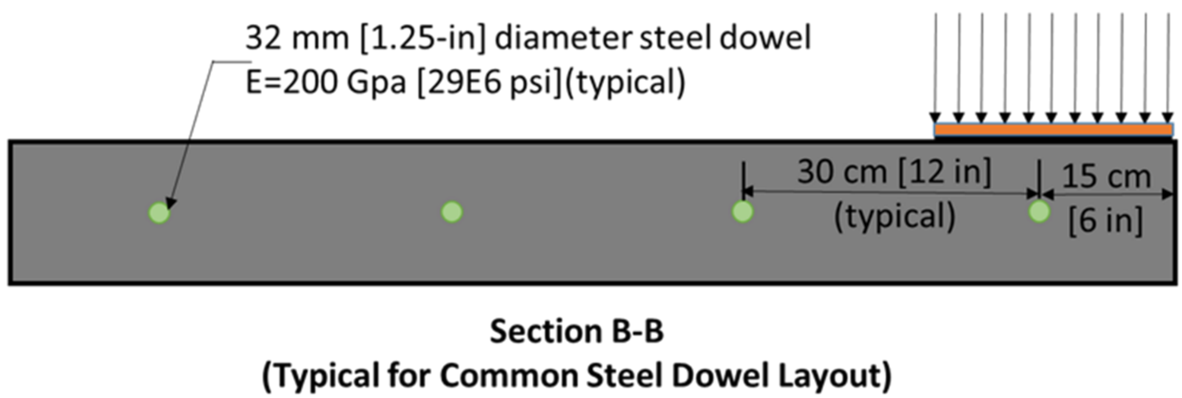

Figure 4. Proposed modified AASHTO T 253 dowel load-deflection test – elevation section B-B

Work was also done in 2016 to establish a relative deflection limit for the new test (for use in specifications) by performing finite element (FE) analyses (using ABAQUS software) of the original and modified AASHTO T 253 tests using 1.25 in. diameter steel dowels and typical material properties for concrete and steel. With all material properties and specimen dimensions being fixed, the only unknown variable that affected relative deflection across the test specimen joints was the modulus of dowel-concrete interaction (K).

The FE analysis was performed iteratively using the same model with several different values of K to produce the graph shown in Figure 5. This figure suggests that a relative deflection of 10 mils corresponds to K of approximately 425,000 psi/in. K cannot be measured directly but has been reported to range between 300,000 and 1.5E6 psi/in., with 1.5E6 being the value most commonly assumed in pavement analyses. At K = 1.5E6 psi/in., the FE model predicted a relative deflection of slightly more than 4 mils.

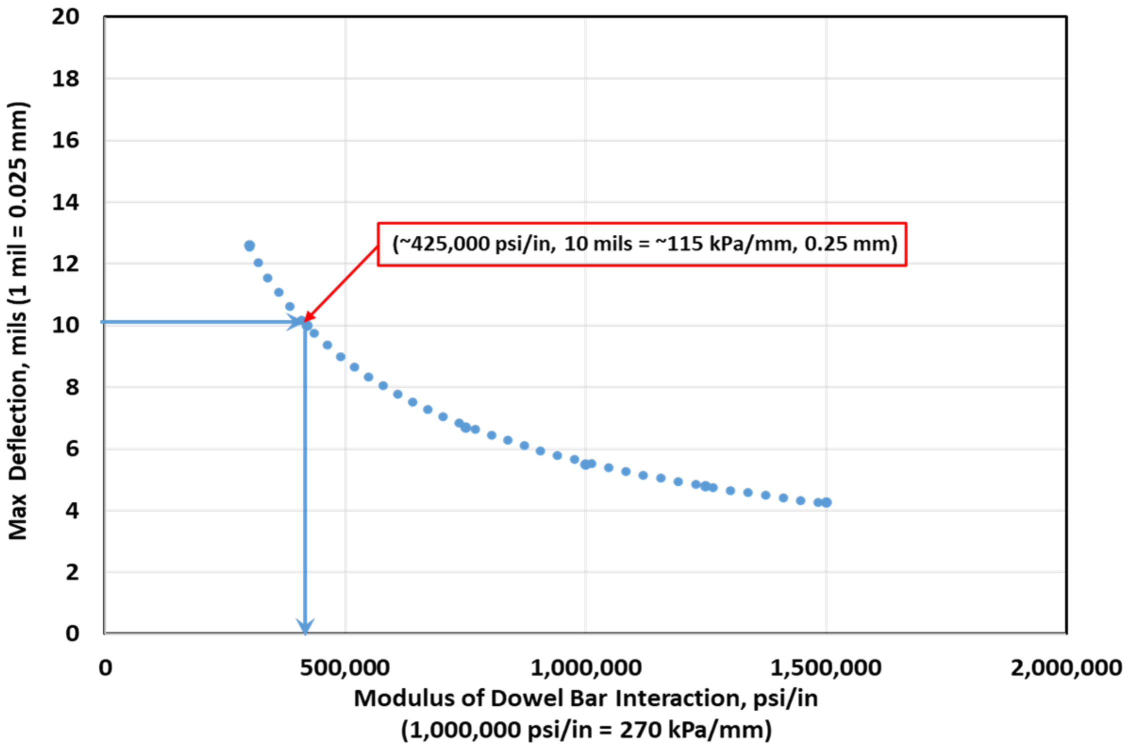

Figure 5. Plot of 2016 FE analysis of AASHTO T 253 load-deflection test for various values of K

Similar FE analysis were performed for the proposed new test configuration, again assuming 1.25 in. diameter steel dowels (on 12 in. centers, as shown in Figure 4), typical concrete and steel material properties, and varying values of K. Figure 6 presents the results of this analysis, which suggests that if the new test is performed using the same dowels and material properties that produced a relative deflection of 10 mils in the current AASHTO T 253 load-deflection test, the predicted relative deflection is approximately 20 mils. At K = 1.5E6 psi/in., the relative deflection is predicted to be about 8.5 mils.

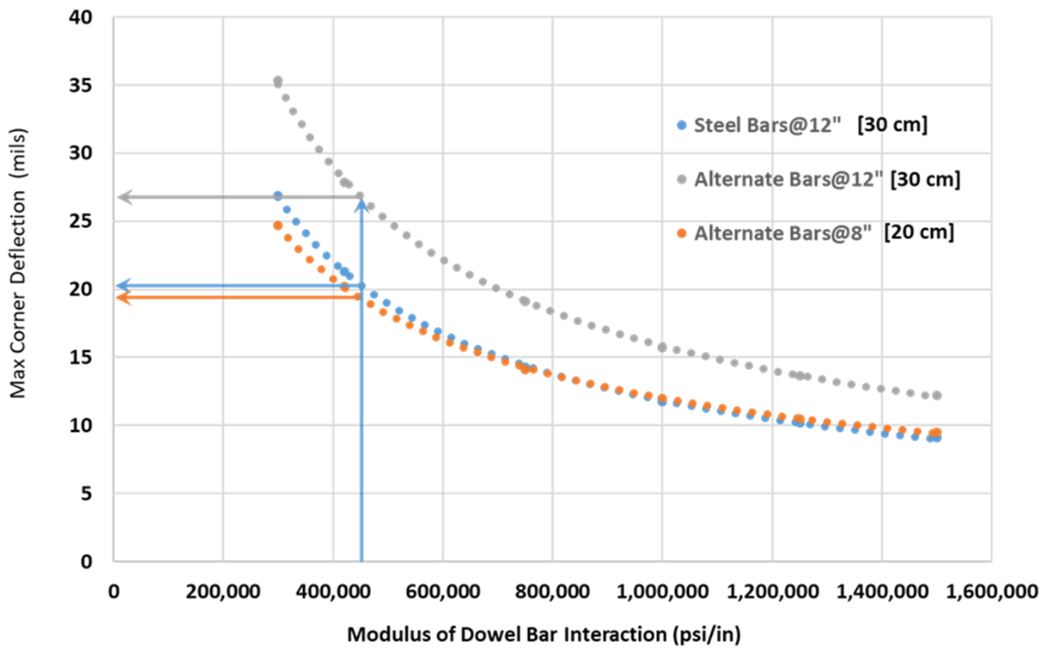

Figure 6. Plot of 2016 FE analysis results for modified AASHTO T 253 load-deflection test for various values of K and various dowel types and spacing

Based solely on this analytical work, one might conclude that a relative deflection limit of 20 mils under the new test protocol would be consistent with the 10 mil limit given in AASHTO M253 for the AASHTO T254 load-deflection test. This conclusion could be validated by performing side-by-side tests using both test protocols and identical materials (including 1.25 in. diameter cylindrical steel dowels). This approach was undertaken in this research study to validate the analytical results and provide guidance in selecting load-deflection limits for implementation in specifications.

TEST PROGRAM

The test program was initially designed as follows:

- Four replicates of AASHTO T 253 load-deflection testing, shown previously in Figure 1:
  - Use 1.25 in. diameter epoxy-coated steel dowels, conventional concrete paving mixture
  - Loading accomplished in compliance with the load rate requirements of AASHTO T 253, Section 5
  - Test program to provide eight separate measures of relative deflection (two replicates for each of four specimens)
- Two replicates of the proposed modified test configuration, shown previously in Figure 2:
  - Use four 1.25 in. diameter epoxy-coated steel dowels per joint, conventional concrete paving mixture (same as for AASHTO T 253 test, cast at the same time from the same batch)
  - Loading accomplished at a rate that results in full load application in the same period of time required to achieve full loading under AASHTO T 253 Section 5
  - Perform load-deflection test in each of four corner test locations per specimen (one in each of the four corners of the unsupported slab), thereby providing a total of eight separate measures of relative deflection
- Cast and test companion cylinders for compressive strength and elastic modulus at time of load-deflection testing

The test procedures were modified (at the request of members of a panel formed by the American Concrete Pavement Association (ACPA) of agency and industry stakeholders who were developing and reviewing improved pavement dowel specifications) to include an overload component in both the original AASHTO T 253 test and the proposed modified test. The purpose of the overload was to gather data on higher test loads and their impacts on dowel behavior under permitted overload conditions. It was believed that such higher test loads might also provide insight into the potential deformation characteristics of some thicker and potentially more compliant coatings (e.g., thick epoxy, layers of FRP).

Therefore, the test protocols were modified as follows:

A load hold test of 10 minutes was applied at the 9,000 lb peak (4,000 lb peak for the original test), and the load was increased to 13,500 lb (6,000 lb for the original test), with another 10-minute load hold at that level. Relative deflection data were collected at the beginning and end of each hold and at 1 minute after unloading.

RESULTS

Appendix A contains the project test program description, test result summaries, and observations of the testing laboratory (Construction Technology Laboratories, Inc.). Appendix B contains raw data collection sheets from the test program.

Tables 1 and 2 present summaries prepared by the present report's author of the test data for the standard AASHTO T 253 and modified test procedures, respectively, and are based on the raw data sheets presented in Appendix B.

Table 1. Load-deflection test results for standard AASHTO T 253 specimens with overload and holds (test date May 21, 2020)

	Load, Hold Time														
	4,000 lb, t = 0			4,000 lb, t = 10 mins			6,000 lb, t = 0			6,000 lb, t = 10 mins			Unload, t = 1 min		
	Relative Deflections, mils (0.001 in)														
SPECIMEN	**J1**	**J2**	**Avg.**	**J1**	**J2**	**Avg.**	**J1**	**J2**	**Avg.**	**J1**	**J2**	**Avg.**	**J1**	**J2**	**Avg.**
1	1.4	3.8	2.6	1.7	4.0	2.9	2.7	5.0	3.9	3.3	5.6	4.5	1.0	1.0	1.0
2	2.9	2.7	2.8	3.0	2.8	2.9	4.1	3.8	4.0	4.4	4.0	4.2	0.9	0.7	0.8
3	2.1	2.0	2.1	2.2	2.1	2.2	3.1	3.1	3.1	3.4	3.4	3.4	0.9	1.2	1.1
4	2.8	4.6	3.7	3.0	5.0	4.0	4.2	6.5	5.4	4.4	6.9	5.7	1.2	5.4	3.3
AVG (1–4)	2.3	3.3	2.8	2.5	3.5	3.0	3.5	4.6	4.1	3.9	5.0	4.4	1.0	2.1	1.5
AVG (1–3)	2.1	2.8	2.5	2.3	3.0	2.6	3.3	4.0	3.6	3.7	4.3	4.0	0.9	1.0	1.0

Table 2. Load-deflection test results for modified AASHTO T 253 specimens with overload and holds (test date May 20, 2020)

		Load, Hold Time																			
		9,000 lb, t = 0				9,000 lb, t = 10 mins				13,500 lb, t = 0				13,500 lb, t = 10 mins				Unload, t = 1 min			
		Relative Deflections, mils (0.001 in)																			
SPECIMEN	**Load Position, Time**	**C1**	**C2**	**C3**	**C4**	**C1**	**C2**	**C3**	**C4**	**C1**	**C2**	**C3**	**C4**	**C1**	**C2**	**C3**	**C4**	**C1**	**C2**	**C3**	**C4**
	1, N/A	**1.5**		-0.9		**1.9**		-0.9		**4.4**		-1.3		**5.3**		-1.3		**3.8**		-0.5	
	2, 9:44a		**3.2**	0.8			**3.4**	0.9			**4.9**	1.7			**5.1**	1.7			**0.4**	0.8	
1	3, 10:44a			**4.0**	3.6			**4.2**	3.7			**5.8**	5.0			**6.2**	5.3			**0.6**	0.1
	4, 11:25a			3.1	**6.3**			3.5	**6.6**			4.7	**8.3**			4.8	**8.8**			0.5	**0.2***
	Avg Loaded		**3.8**				**4.0**				**5.9**				**6.4**				**1.6**		
	1, 2:13p	**2.6**	5.9			**2.9**	6.5			**5.4**	8.3			**6.0**	8.9			**1.1**	3.1		
	2, 2:53p	3.7	**6.0**			3.8	**6.5**			4.9	**9.0**			5.0	**9.8**			0.5	**1.2**		
2	3, 3:31p		-2.3	**6.6**			-2.7	**7.3**			-3.6	**9.4**			-3.8	**10.2**			-1.1	**2.6**	
	4, 4:09p		-0.6		**2.7**		-0.8		**2.7**		-1.2		**4.3**		-1.2		**4.3**		-0.3		**-0.8**
	Avg Loaded		**4.5**				**4.9**				**7.0**				**7.6**				**1.0**		

Enhanced font indicates deflection under applied load.
* Transducer moved

AASHTO T 253 Load-Deflection Test Data

Figure 7 presents a plot of the relative deflection progression for each joint of each specimen (e.g., S1-J1 is data for Specimen 1, Joint 1). The data are extracted from Table 1.

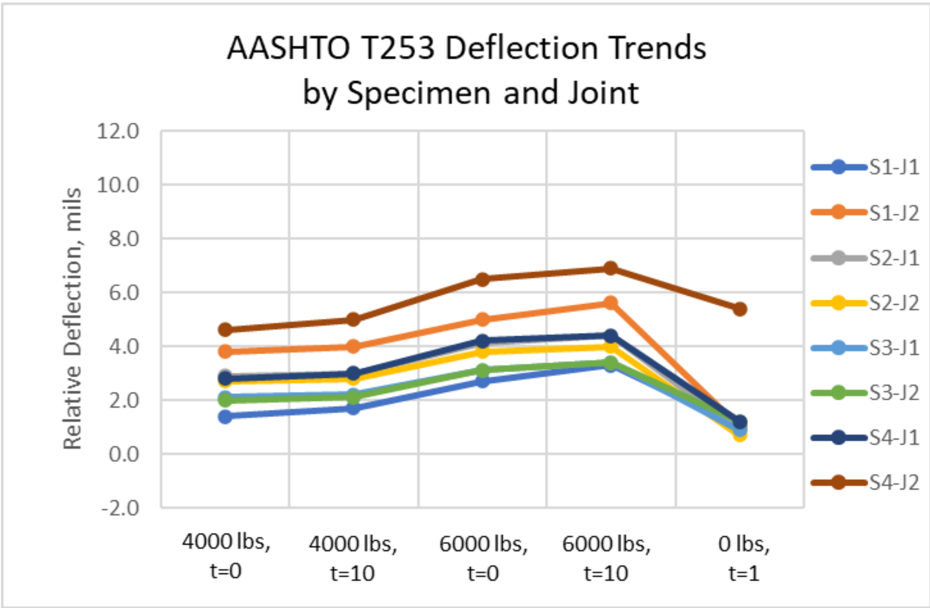

Figure 7. Plot of relative deflection data for each joint of AASHTO T 253 load-deflection test specimen at each stage of test (t = 0 minutes)

Joint 2 of Specimen 4 exhibits unusually high deflections throughout the entire test, including very high residual deflection after the load has been removed at the end of the test. Possible reasons for this behavior include (1) a void or other weakness around the dowel in S4-J2 and (2) movement or incorrect zeroing of the deflection measuring device.

In any case, it appears that data from this specimen are unreliable and should be disregarded. Therefore, Table 1 includes one summary line that includes the results of all four specimens and one line that represents only Specimens 1 through 3; the latter was used for comparisons and analyses in this report.

The load-deflection behavior of all specimens is generally as expected, with significant deflection increases (or decreases) at each load increment (or decrement) and slight deflection increases during load hold periods. Small residual relative deflections (0.7 to 1.2 mils for the joints in Specimens 1 through 3) remain a short time after the load is removed.

The average relative deflection for Specimens 1 through 3 after 4,000 lb loading (in compliance with AASHTO T 253 Section 5) is 2.5 mils, well below the 10 mil AASHTO M 254 limit. This low relative deflection value suggests that the modulus of dowel-concrete interaction, K, was much higher than the assumed value of 425,000 psi/in. shown in Figure 5 (which would, in theory, have produced 10 mils of relative deflection).

Figure 8 presents a graphical extrapolation of the finite element analysis data previously presented in Figure 5 and indicates that the effective or apparent K for these tests is approximately 2.5 million psi/in., which is outside the range of typically assumed values for K. This suggests that there may be problems with the model or the test; these possibilities are discussed later in this report.

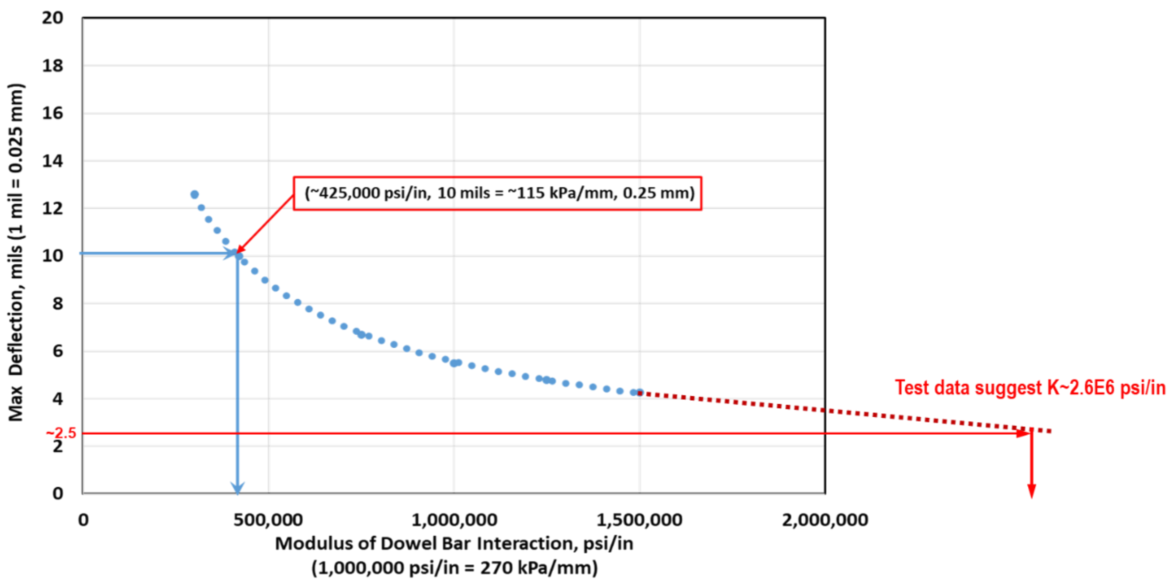

Figure 8. Extrapolation of Figure 5 data to estimate experimental K for AASHTO T 253 load-deflection test

Modified AASHTO T 253 Load-Deflection Test Data

Figures 9 and 10 present plots of the relative deflection data for each load position for Specimens 1 and 2, respectively; data series are labeled by specimen and load position (e.g., S1-LP1 represents data for Specimen 1, Load Position 1). The data are extracted from Table 2.

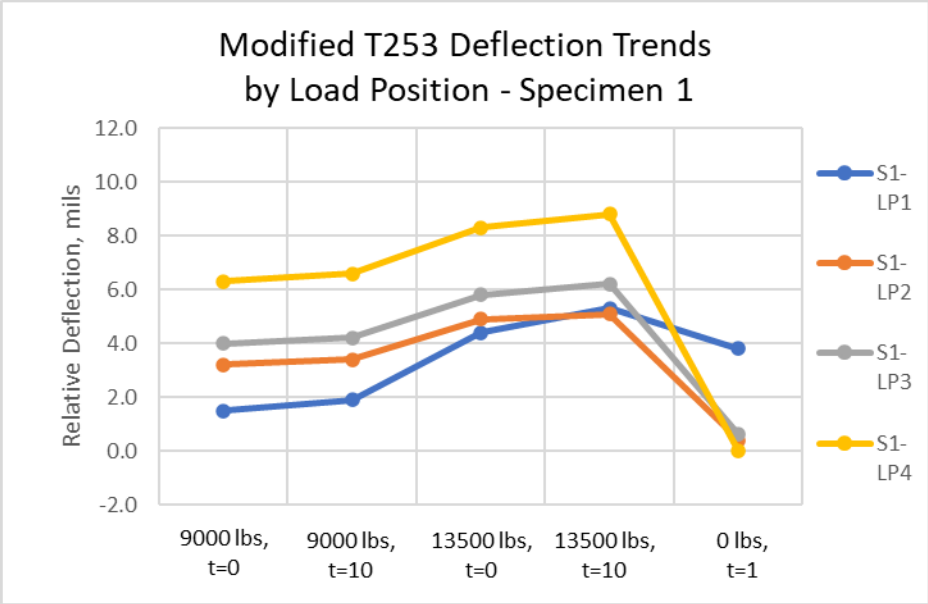

Figure 9. Plot of relative deflection data for each load position of modified AASHTO T 253 load-deflection test Specimen 1 at each stage of test protocol (t = 0 minutes)

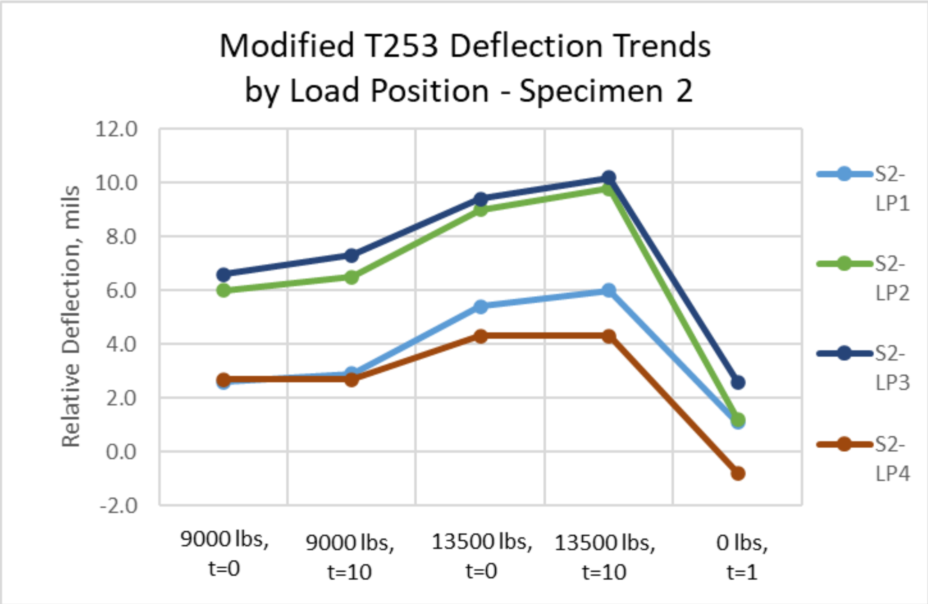

Figure 10. Plot of relative deflection data for each load position of modified AASHTO T 253 load-deflection test Specimen 2 at each stage of test protocol (t = 0 minutes)

The load-deflection behavior at all load positions for both modified specimens is generally as expected, with significant deflection increases (or decreases) at each load increment (or decrement) and slight increases during load hold periods. Small residual relative deflections remain a short time after the load is removed in most cases. The residual deflection for Specimen 1, Load Position 1 is unusually high, which may indicate that the sensor moved when the load was released.

Figures 9 and 10 indicate that *a load sequence bias exists in the data for each specimen*: the deflections at the first load position are lowest, the deflections at the second load position are slightly higher, the deflections at the third load position are higher still, and the deflections at the fourth position are highest of all (for Specimen 1 but not Specimen 2).

The load-deflection profile for the fourth load position of the second specimen seems to resemble the profile for the first load position, except that the residual deflection is less than zero, which is counterintuitive and suspect. A possible explanation for this profile is that the deflection-measuring device slipped or was not correctly zeroed at the start of testing. If the entire profile is translated vertically on the graph such that the residual deflection after test completion is about the same as was observed for Load Positions 2 and 3, the entire curve would be positioned more consistently with Load Position 4 for Specimen 1.

There are two potential reasons for the apparent load sequence bias: (1) movement of the specimen after each test to position the test locations for Load Positions 2, 3 and 4 under the load actuator (which was in a fixed location) produced some dowel looseness or localized damage that accumulated with each move, or (2) each load sequence induces some loading of all dowels in the specimen (not just the one directly under the load), which results in some looseness or localized damage at all dowels that shows up as increased relative deflections in subsequent tests. The potential effects of load-induced damage are discussed later in this report. Additional testing is required to determine whether specimen handling also contributed to the apparent test result bias.

Because of the apparent test sequence bias in this data set, only the data from the first load position of each specimen are considered in this report for comparison with data from the standard AASHTO T 253 load-deflection test. These data lead to the following observations (summarized in Table 3):

- At the initial load of 9,000 lb, the average relative deflection is (2.6 + 1.5) / 2 = 2.05 mils (only for Load Position 1 of both specimens), which can be compared with the 2.5 mil average initial deflection observed for the standard AASHTO T 253 Section 5 load-deflection test at 4,000 lb.
- At the increased load of 13,500 lb, the average relative deflection is (4.4 + 5.4) / 2 = 4.9 mils (only for Load Position 1 of both specimens), more than double the 2.05 mil average deflection at 9,000 lb. This can be compared with the 3.6 mil average deflection observed for the standard AASHTO T 253 Section 5 load-deflection test at 6,000 lb, which is almost 50% higher than the 2.5 mil deflection observed at 4,000 lb.

Table 3. Summary of load-deflection comparisons between standard and modified AASHTO T 253 tests at standard and 150% load

	Standard Load	150% Load
Standard AASHTO T 253	2.5 mils	3.6 mils
Modified AASHTO T 253	2.05 mils	4.9 mils

Note: Standard and 150% loads for standard AASHTO T 253 are 4,000 and 9,000 lb, respectively. Standard and 150% loads for modified AASHTO T 253 are 9,000 and 13,500 lb, respectively.

The data in Table 3 lead to the following observations:

- The modified AASHTO T 253 test deflection was about 20% lower than the standard AASHTO T 253 test deflection under "standard" load conditions but about 36% higher under 150% load conditions.
- The standard AASHTO T 253 deflections increased approximately linearly with increased load. The modified AASHTO T 253 deflections increased nonlinearly; a 50% increase in load produced a 139% increase in deflection.
- The finite element models greatly overpredicted deflections for both test protocols. The following sections describe investigations and analyses into the observations above.

DISCUSSION AND DATA ANALYSES

Test Data

Evaluation and interpretation of the load-deflection data trends for the two test protocols requires (1) an understanding of the shear loads being applied to and transferred through the critical dowels and (2) consideration of the effects of slab stiffness on load distribution and on restraining relative deflection at the measurement location.

Critical Dowel Shear Load

Determining the shear loads in the standard AASHTO T 253 test is simple because there is only one dowel at each end of the loaded and unsupported center section of the test specimen, so each dowel carries ½ of the applied load (i.e., 2,000 lb/dowel for the standard 4,000 lb load application and 3,000 lb/dowel for the 6,000 lb load application). Determining dowel loads for the modified AASHTO T 253 load-deflection test procedure is more complicated.

Determination of the critical dowel load in response to a corner load for a grade-supported doweled joint can be performed by distributing the load linearly to the affected dowels within the radius of relative stiffness (measured from the point of load application). This computation is described in Appendix B of the National Concrete Pavement Technology Center (CP Tech Center) publication *Guide to Dowel Load Transfer Systems for Jointed Concrete Roadway Pavements*, which is reproduced as Appendix C to this report.

The example calculation presented in Appendix C assumes a 9,000 lb applied load over the dowel closest to the slab edge, just as was done in the modified AASHTO T 253 testing. It also assumes a 10 in. slab thickness, 12 in. dowel spacing, $E_{concrete}$ = 4.0E6 psi, and Poisson's ratio = 0.17, all of which are consistent with (or reasonable assumptions for) the modified AASHTO T 253 test performed for this study. Assumptions that do not conform with the modified AASHTO T 253 test are the assumed subgrade modulus of 200 psi/in. (a reasonable effective level of soil support for many field conditions, but there is no support of the center panel in the modified AASHTO T 253 test) and the transferred load percentage of 42% (100% of the load must be transferred from the unsupported slab to the two support slabs, with a greater percentage likely going through the dowels in the joint closest to the applied load).

The example presented in Appendix C for grade-supported joint systems estimates the critical dowel load at 1,881 lb for a 9,000 lb edge load condition, about 95% of the 2,000 lb shear load induced in each dowel in the original AASHTO T 253 load-deflection test. The 50% overload conditions (6,000 lb for the standard AASHTO T 253 test and 13,500 lb for the modified AASHTO T 253 test) would likely result in linearly scaled (for these load and support conditions) dowel shear loads (i.e., 3,000 lb for the standard test and 2,822 lb for the modified test). The AASHTO T 253 test condition is not grade supported, however, so these dowel shear load estimates for the modified test are not realistic.

ABAQUS finite element software was used to model the theoretical deflection and dowel shear load distribution for the modified AASHTO T 253 test over a range of values for K (modulus of dowel-concrete interaction). The results of these analyses are presented in Table 4, and a graphical representation (exaggerated) is shown in Figure 11. In Table 4, Bars 1 through 4 are located along the joint closest to the load (Bar 1 is directly under the load), and Bars 5 through 8 are located along the joint farthest from the load (Bar 5 is aligned with Bar 1, Bar 6 with Bar 2, etc.).

Table 4. Tabulation of modified test dowel shear loads for 9,000 lb applied load (ABAQUS model)

K	Vertical Dowel Shear Force for 9,000 lb Load (New Model), lb								Check
(psi/in.)	Bar 1	Bar 2	Bar 3	Bar 4	Bar 5	Bar 6	Bar 7	Bar 8	Sum
300,000	-3,707	-2,335	-979	375	-2,621	-1,267	84	1,422	-9,027
600,000	-3,718	-2,338	-981	375	-2,614	-1,262	90	1,419	-9,028
900,000	-3,725	-2,339	-981	376	-2,610	-1,259	93	1,415	-9,029
1,200,000	-3,731	-2,339	-981	378	-2,607	-1,256	95	1,411	-9,029
1,500,000	-3,736	-2,339	-980	380	-2,604	-1,254	96	1,408	-9,030
1,800,000	-3,740	-2,339	-980	381	-2,602	-1,253	98	1,404	-9,030
Average	-3,726	-2,338	-980	377	-2,609	-1,258	93	1,413	-9,029

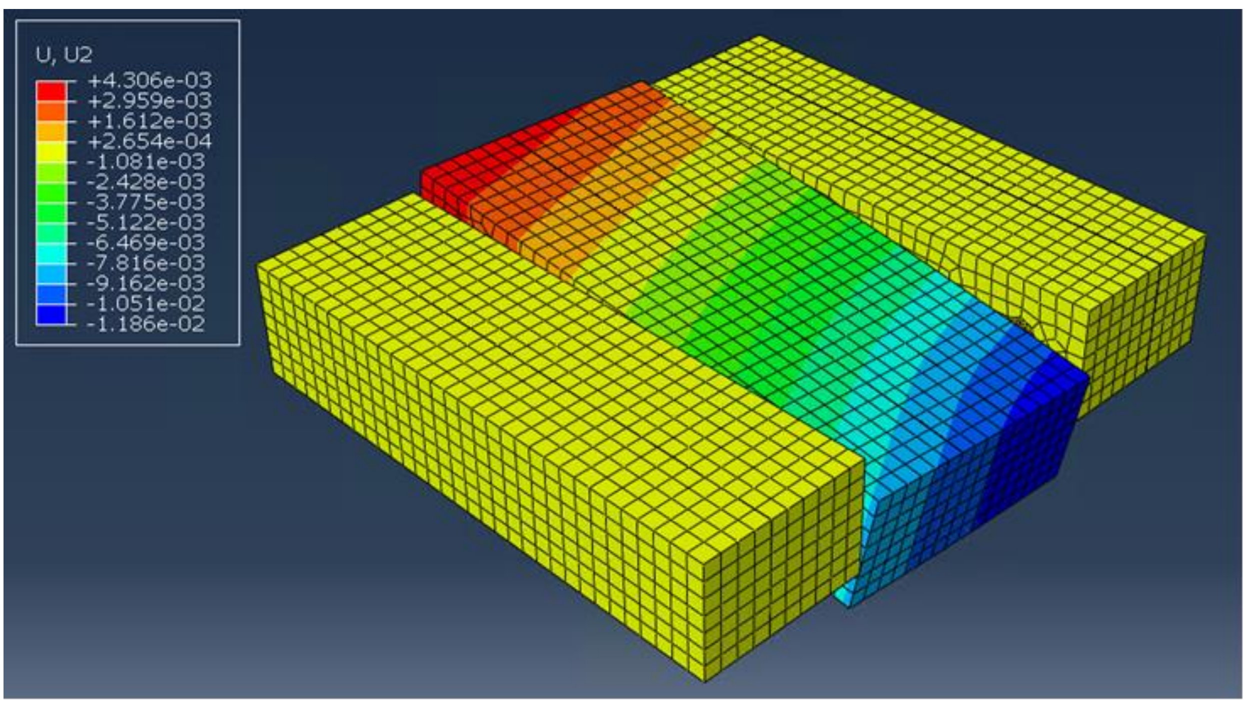

Figure 11. ABAQUS model of slab deflections for modified AASHTO T 253 test under 9,000 lb corner load

Table 4 and Figure 11 show that the modified test produces a much greater load on the critical dowel, approximately 3,700 lb for a 9,000 lb applied load (and likely more than 5,500 lb for a

13,500 lb applied load). This increased critical load is due to the asymmetrical load location in a slab corner (rather than the linear or uniformly distributed load in the standard AASHTO T 253 test), which results in rotation of the slab about both the X and Y axes and corresponding differences in dowel shear load, including slab uplift and negative loads around the opposite corner from the load application. This slab uplift was observed in the CTLGroup laboratory test program (see, for example, the raw data collection sheets for "Loading at Corner 1-1" and "Loading at Corner 2-4" in Appendix B).

Figure 12 presents data plots of transferred load versus computed Friberg bearing stress for solid 1.25 in. diameter steel dowels (E = 29E6 psi, blue lines) and low-modulus dowels (E = 6E6 psi, green lines) for two values of dowel-concrete interaction (K = 5E5 and 1.5E6 psi/in., dashed and solid lines, respectively). The analysis indicates that the bearing stress for the standard AASHTO T 253 test with K=1.5E6 psi/in. (a typically assumed value) is approximately 2,160 psi (increasing to 3,240 psi for the 50% overload condition), while the bearing stress for the modified AASHTO T 253 test is 3,995 psi (increasing to nearly 6,000 psi for the 50% overload condition). The low-modulus dowel bearing stresses for the same four conditions are 3,360, 5,040, 6,220, and 9,320 psi. ACI 325 (ACI Committee 325 1956) would limit bearing stress to 92% of f'c. CTLGroup reported the average compressive strength of the concrete companion specimens as 6,350 psi at the time of testing, so bearing stress would be limited to 5,820 psi. This value was exceeded during the testing at overload conditions and may have caused the development of some concrete microfracturing and dowel looseness that contributed to the test sequence bias that was observed with the modified AASHTO T 253 test.

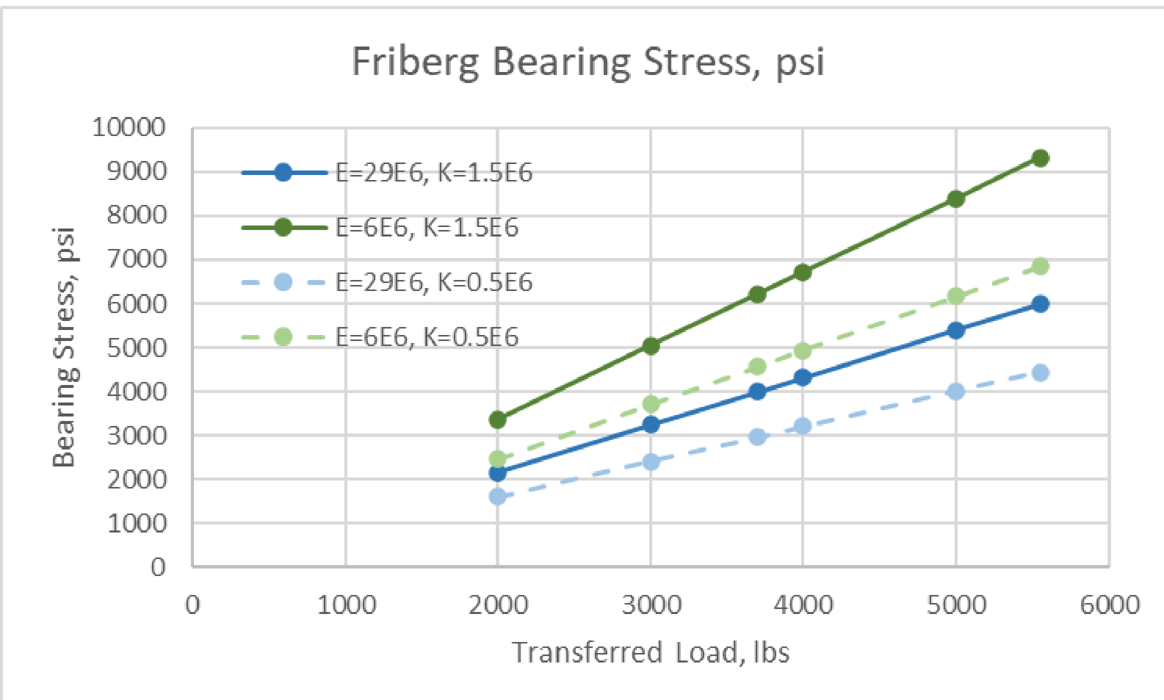

Figure 12. Calculated bearing stresses for critical dowel in modified AASHTO T 253 load-deflection test for a range of transferred shear loads and selected dowel modulus and K values

As a point of reference, if the compressive strength was a nominal 4,000 psi (typical portland cement concrete pavement [PCCP] design strength), ACI 325 (ACI Committee 325 1956) would limit bearing stress to 3,680 psi, which is greater than was computed for either the standard AASHTO T 253 test (acceptable) but lower than computed for even the 9,000 lb applied load with the modified test (a potential problem).

Effects of Slab Stiffness

The test results shown in Table 3 indicate that the modified test scheme with a 9,000 lb load produced about 20% less relative deflection than the standard AASHTO T 253 test scheme with a 4,000 lb load, even though the FE models indicate a much higher shear load in the critical dowel. (The FE models also indicate that the modified test should produce higher deflections than the original test. This is discussed in the next section of this report.)

The discussion in the previous section showed that dowel-concrete bearing pressures at the standard 4,000 and 9,000 lb loads are approximately at or below the ACI 325 (ACI Committee 325 1956) limits, suggesting that the concrete is not being heavily damaged by dowel-concrete bearing stresses in either test configuration at the lower load levels. The primary source of resistance to deflection in the current AASHTO T 253 load-deflection test is dowel stiffness; the stiffness of the uniformly loaded concrete slab likely contributes little to the relative deflection behavior at the joints. The same is not true for the modified AASHTO T 253 test.

The modified test deflections at the point of measurement are likely restrained by the presence of adjacent dowels and the stiffness of the slab connecting the critical (under the load) dowel with those adjacent dowels. In other words, the observed deflections in the modified load test are likely lower than would have been observed if the same 3,700 lb (estimated) shear load were applied to a single dowel. Current AASHTO T 253 test data confirm this by comparing the 2,000 and 3,000 lb shear load data; one might project that the deflection would have been approximately 4.6 mils at a 3,700 lb shear load. Therefore, deflections at the critical dowel will be restrained in the modified test until loads increase to a point where the adjacent dowel deflections also increase enough to allow additional deflection at the critical dowel.

This may be why the higher load increment in the modified test exhibits such a great increase in deflection (>150%); the critical dowel has a very high load (perhaps producing bearing failure) and the adjacent dowel load is also quite high, allowing more bending of the slab between the two dowels.

Modeling

The test data presented in Table 3 (i.e., 2.5 mils relative deflection for the 4,000 lb AASHTO T 253 test and 2.05 mils for the 9,000 lb modified AASHTO T 253 test) do not match the values predicted by the original finite element models for any reasonable value of K (see Figures 5 and 6), which show predicted relative deflection values of about 4 mils and 8.5 mils for the standard and modified AASHTO T 253 tests, respectively, for K = 1.5E6 psi/in. (a commonly assumed value of K). There are at least three possible explanations for the discrepancy: (1) K is much greater than 1.5E6 psi/in. for these tests (a possibility that is not supported by experimental tests in the literature); (2) the test procedures and measurements are flawed (always a possibility, but the repeated measurements were reasonably consistent); and (3) the original models were flawed. The analyses and discussion presented below describe an investigation into the third possibility.

Copies of the original ABAQUS model files for both the original and modified AASHTO T 253 test setup were obtained from the original modeler. These files were uploaded into a recent version of ABAQUS software, and the model setup and input parameters were compared for consistency and accuracy. Figure 13 presents isometric and plan view depictions of the AASHTO T 253 test model to illustrate the types of model components (cylindrical dowel segments, "donut" segments for concrete immediately surrounding the dowels, block segments for bulk concrete and end sections, and gaps for the joints). The same types of elements were used to assemble the model for the modified AASHTO T 253 test.

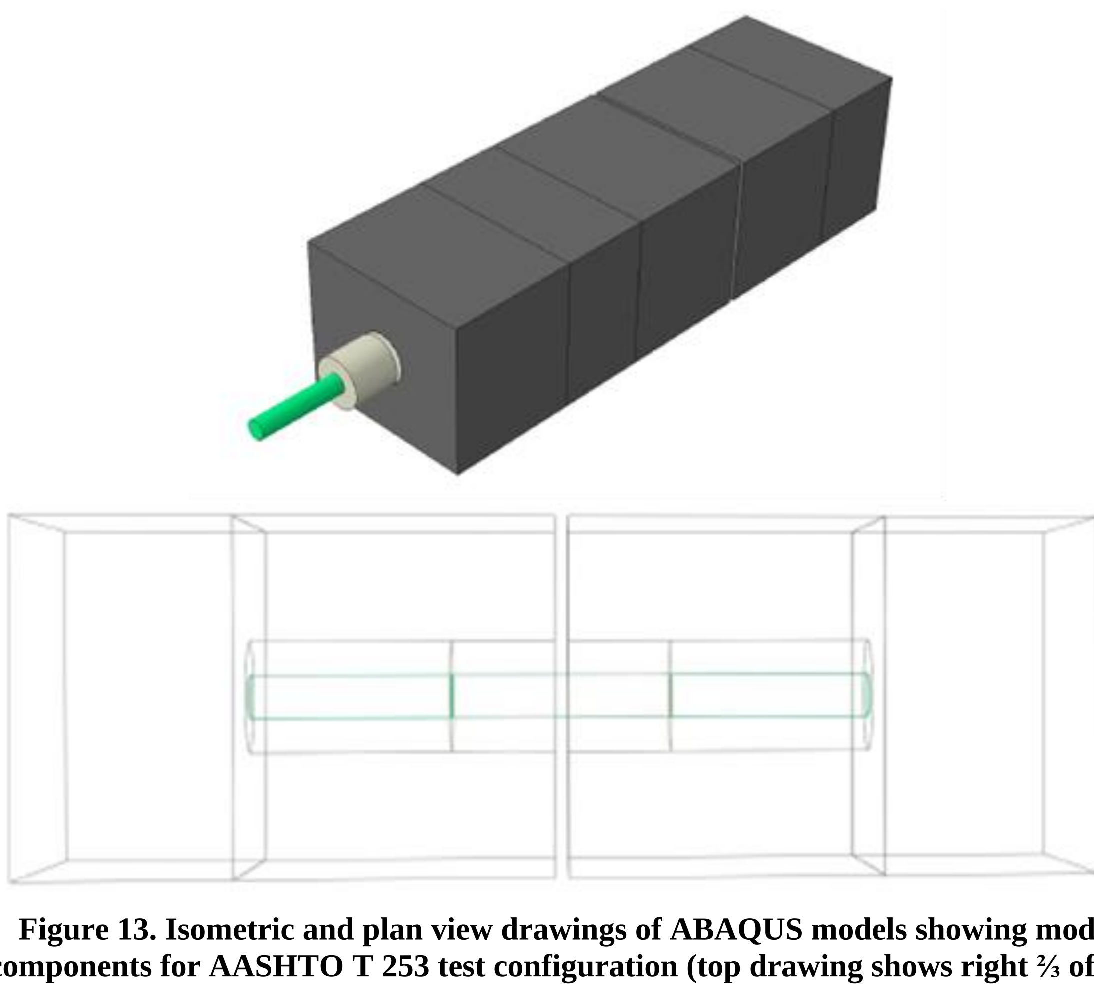

Figure 13. Isometric and plan view drawings of ABAQUS models showing model components for AASHTO T 253 test configuration (top drawing shows right ⅔ of full specimen; bottom shows ½ of full specimen)

The following potential issues were identified:

- The load used in the original AASHTO T 253 model file was double what it should have been. It is possible that the file parameters reflected a sensitivity test run that was saved and that the actual value used to develop the plot in Figure 5 was correct.
- The elastic modulus of the dowel in the original AASHTO T 253 model file was set at 11E6 psi rather than 29E6 psi. It is possible that the file parameters reflected a sensitivity test and that the actual value used to develop the plot in Figure 5 was correct.
- The model of the original AASHTO T 253 test designated the "donut" region surrounding the concrete as "damaged concrete" that would deform nonlinearly with increasing stress. The model of the modified test treated the "donut" region as concrete with linear elastic behavior and providing support to the dowel as a spring interaction (compression only, no tension).
- The original AASHTO T 253 test model divided the dowel and "donut" region into several segments (with breaks at every model boundary, including dowel ends and joint faces). The modified AASHTO T 253 test model was greatly simplified with only three segments per dowel, including one that bridged the joint completely for each dowel.
- The meshing used in each model was judged to be suboptimal, with some unnecessarily elongated and/or flat elements and some mismatched nodes (resulting from rapid changes in mesh fineness between elements).

Both models were revised and refined to address the potential deficiencies identified. Revisions included the following:

- Use of the correct load (4,000 lb) and dowel elastic modulus (29E6 psi) in the model of the AASHTO T 253 load-deflection test.
- Replacement of the "damaged concrete" elements with linear elastic concrete and spring interaction support of the dowel (compression only) in the AASHTO T 253 load-deflection test model.
- Use of the simplified three-component dowel modeling in both models.
- Remeshing of both models to improve element geometry and improve mesh fineness transitions to reduce the incidence of mismatched nodes.

Finite element analyses were run over a range of K values for both models and standard load conditions (i.e., 4,000 lb for the original AASHTO T 253 test model and 9,000 lb for the modified test model). Table 5 and Figure 14 present tabular and graphical summaries of the predicted relative deflections for the original and updated models for the AASHTO T 253 test.

Table 5. Comparison of predicted relative deflections versus K for AASHTO T 253 load-deflection test using updated and original FE models

	Relative Deflection, mils	
K (psi/in.)	Updated Model	Original Model
300,000	7.94	16.44
600,000	5.08	9.94
900,000	3.98	7.55
1,200,000	3.37	6.29
1,500,000	2.98	5.51
1,800,000	2.71	4.97

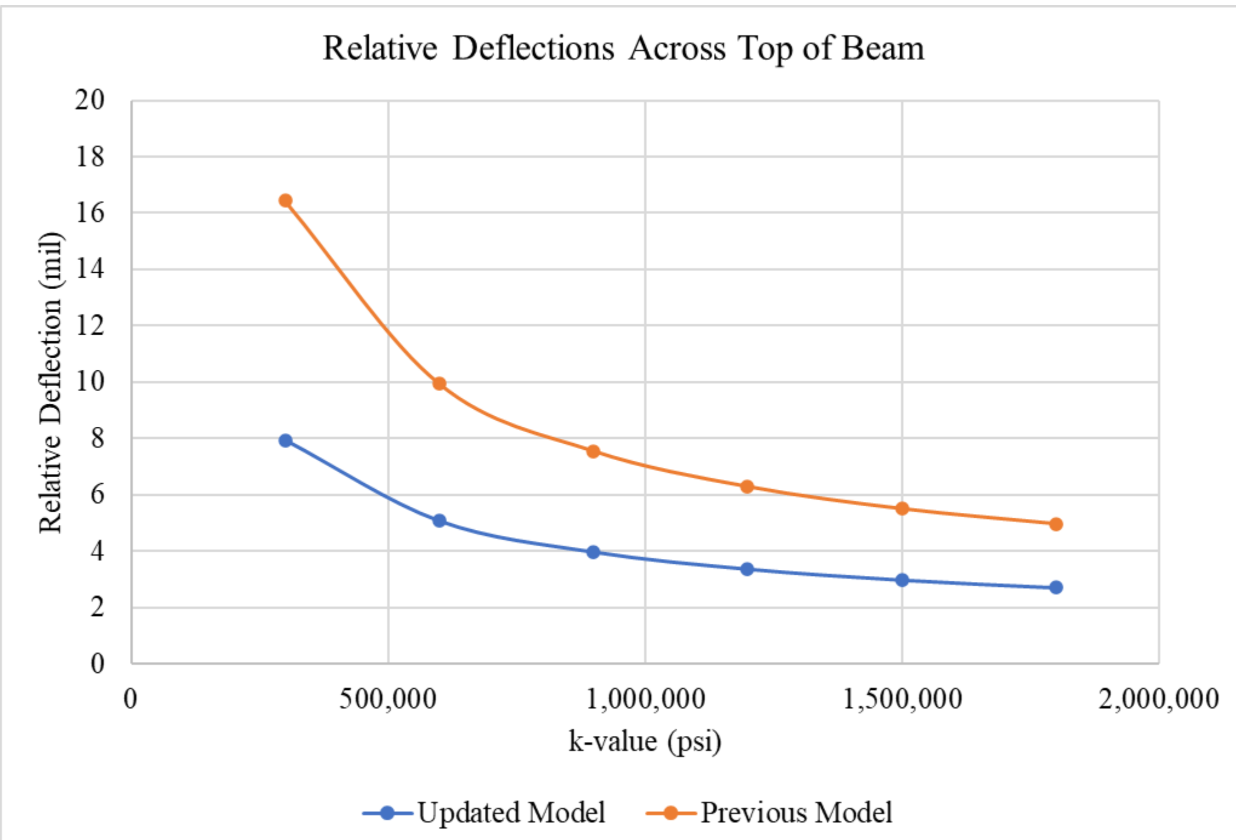

Figure 14. Graphical presentation of data from Table 5

Recalling that CTLGroup's average result for the AASHTO T 253 tests was 2.5 mils and that a typical assumed value for K is 1.5E6 psi/in., the updated model appears to slightly overestimate measured deflections (2.98 mils versus 2.5 mils) but is far better than the original model prediction (5.51 mils).

An additional analysis run was performed using the updated AASHTO T 253 model with K = 1.5E6 psi/in. and with the load increased to 6,000 lb. The estimated average relative deflection was 4.55 mils, which is approximately 50% higher than predicted for 4,000 lb of loading (as expected) and again slightly overestimates the average measured relative deflection of 3.6 mils.

Table 6 and Figure 15 present the results of similar finite element analyses using the updated model for the modified AASHTO T 253 load-deflection test. Recalling that CTLGroup's average result for the modified AASHTO T 253 tests was 2.05 mils (first test location only) and that a typical assumed value for K is 1.5E6 psi/in., the updated (simplified) model comes closer than the original model (6.91 mils versus 9.52 mils) but still greatly overestimates measured test results.

Table 6. Comparison of predicted relative deflections versus K for modified AASHTO T 253 load-deflection test using updated and original FE models

	Simplified Model – Relative Deflections (in.)							
K (psi/in.)	**Bar 1**	**Bar 2**	**Bar 3**	**Bar 4**	**Bar 5**	**Bar 6**	**Bar 7**	**Bar 8**
300,000	22.76	14.33	5.97	-2.34	16.19	7.85	-0.44	-2.34
600,000	13.75	8.84	3.67	-1.43	9.95	4.81	-0.28	-1.43
900,000	10.19	6.69	2.78	-1.08	7.52	3.63	-0.21	-1.08
1,200,000	8.21	5.51	2.28	-0.89	6.19	2.98	-0.18	-0.89
1,500,000	6.91	4.75	1.97	-0.77	5.33	2.56	-0.15	-0.77
1,800,000	5.98	4.22	1.74	-0.68	4.73	2.27	-0.14	-0.68

	Original Model – Relative Deflections (mils)							
K (psi/in.)	**Bar 1**	**Bar 2**	**Bar 3**	**Bar 4**	**Bar 5**	**Bar 6**	**Bar 7**	**Bar 8**
300,000	24.17	14.59	5.74	-2.56	16.41	7.74	-0.44	-8.38
600,000	15.74	9.30	3.55	-1.67	10.42	4.81	-0.29	-5.16
900,000	12.46	7.24	2.69	-1.32	8.09	3.69	-0.23	-3.92
1,200,000	10.66	6.12	2.23	-1.13	6.83	3.08	-0.20	-3.25
1,500,000	9.52	5.40	1.94	-1.01	6.02	2.69	-0.17	-2.82
1,800,000	8.72	4.90	1.73	-0.92	5.47	2.42	-0.16	-2.53

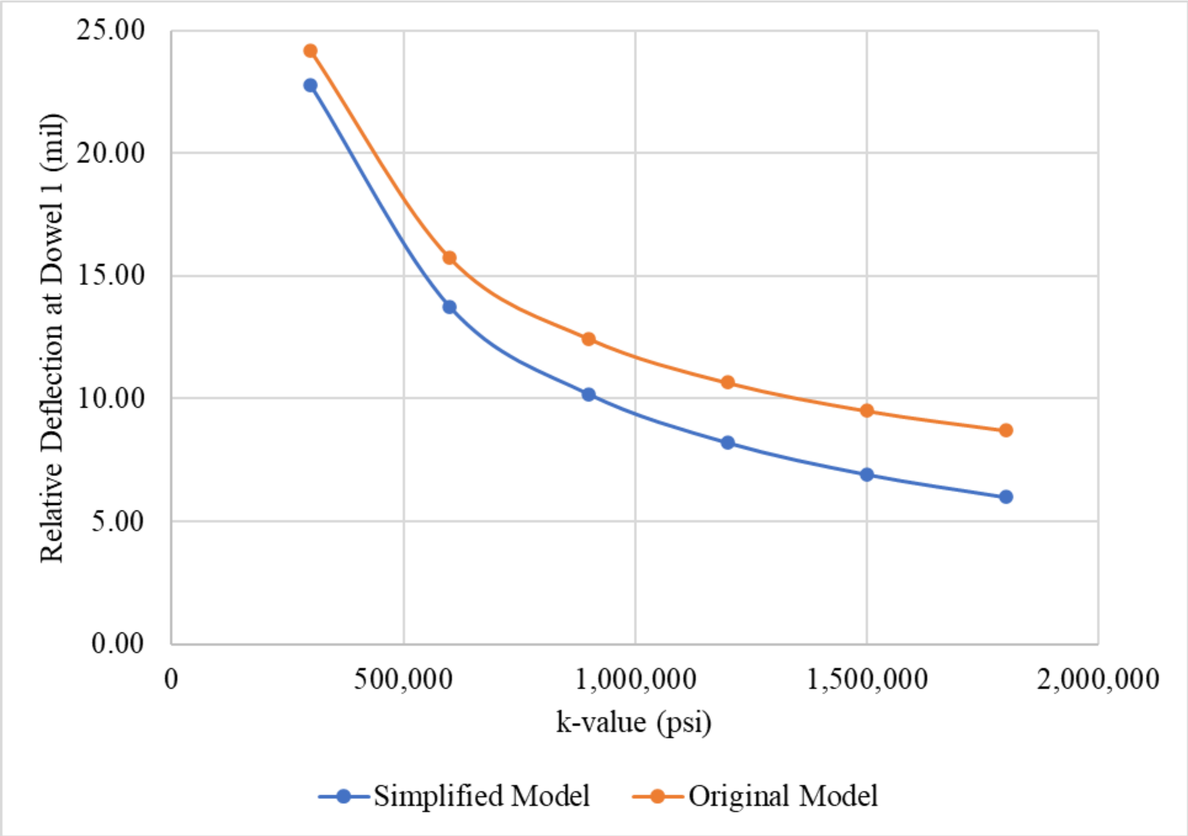

Figure 15. Graph of relative deflection versus K for the modified AASHTO T 253 load-deflection test for the original and updated finite element models (data for Dowel Bar 1 only [under the applied load] from Table 6)

Modeling the Test Procedure

There is one difference between the actual test conditions and the conditions modeled in the finite element program: support of the two end slabs or blocks in each test. The finite element model assumes an absolutely rigid foundation; the slabs tested at CTLGroup were placed on a 1 in. thick layer of plywood to provide uniform support of the slab on the laboratory floor (assuming that the slab bottoms might have some irregularities from casting and that the floor might not be perfectly level and planar). The plywood support of the end blocks can be seen in Figures 16 and 17.

Figure 16. Photo of AASHTO T 253 test setup in CTLGroup laboratory

Figure 17. Photo of modified AASHTO T 253 test setup in CTLGroup laboratory

The potential effects of this plywood layer on relative deflection measurements were considered during the test program planning and were considered negligible because (1) the compressive pressure in the plywood would be low (distributed over a large area by the concrete blocks) and (2) any plywood deformation that resulted in deflection of the fixed end blocks would simply increase the total deflection of the unsupported center block as well, with the relative deflection remaining approximately constant.

It is possible that the compression of the plywood at the joint edge allowed enough joint rotation to affect the measurement of relative deflection. Additional specimen casting and testing on steel plates could be performed to determine whether plywood compression somehow reduced measured relative deflections.

CONCLUSIONS

- Load-deflection testing performed using the AASHTO T 253 setup produced reasonably repeatable results under all loading conditions.
- Load-deflection testing performed using the modified AASHTO T 253 setup exhibited test sequence bias with increasing deflection measurements at each succeeding load position for both specimens. Finite element analysis indicates that the proposed load configuration and magnitude likely result in excessive dowel loads and bearing stresses (nearly double those of the standard AASHTO T 253 test) that induce increasing amounts of dowel looseness with each successive load application. Therefore, only data from the first load application on each modified test specimen were used for test validation purposes in this report.
- Another potential contributor to the apparent test sequence bias is the movement of the specimens during testing (i.e., repositioning of the specimen rather than repositioning of the load actuator).
- The original 2016 finite element models greatly overpredicted observed relative deflections for both test configurations. Modifications that were made to the models in 2021 greatly improved their apparent accuracy, especially for the original AASHTO T 253 test configuration.
- Remaining differences between model predictions and actual test values may be due (at least in part) to the use of thick plywood support layers in the testing laboratory to eliminate irregularities and nonuniform contact between the specimen bottoms and the support floor.

RECOMMENDATIONS

- Modified AASHTO T 253 testing should be reconfigured to develop approximately the same shear load transfer in the critical dowel as is present in the standard AASHTO T 253 test (i.e., 2,000 lb). This would be accomplished with a base load of (2,000 / 3,736) * 9,000 ~ 4,800 lb for the modified test (and approximately 7,200 lb for the optional increased load level).
  - Adopting the 4,800 lb load for the modified test procedure to produce a 2,000 lb load in the critical dowel (like in the standard AASHTO T 253 procedure) would justify the use of the same load-deflection threshold as the current AASHTO T 253 test (i.e., 10 mils or 0.25 mm) and would eliminate the potential for overstressing the concrete, which was likely a major cause (possibly the only cause) of the observed test sequence bias. Eliminating this bias would provide four data points per test specimen rather than the single data point found useful in this experiment.
  - Laboratory testing should be repeated with this reduced load level to confirm the expected behavior.
- Future testing should be performed on a truly rigid foundation (i.e., steel or concrete rather than plywood over concrete) to determine whether the plywood is in any way responsible for deflection measurements that were lower than predicted by the updated finite element models.
- Future modified AASHTO T 253 testing should be performed by moving the actuator to the different load positions rather than repositioning the specimen under a fixed actuator position. This would eliminate the possibility that specimen handling during repositioning increases dowel looseness and relative deflections during load testing.
- After the modified test protocol (i.e., reduced load, rigid foundation, no specimen movement) has been validated, testing should be expanded to include 1.5 in. diameter epoxy-coated steel dowels (which are more common than the 1.25 in. dowels used in the standard AASHTO T 253 test).
- Testing should also be expanded to include tests of specimens containing dowels with alternate materials and structural configurations that have been tested under dynamic load-deflection test protocols (e.g., tubular steel dowels, FRP dowels, etc.). This would help to further validate the proposed test and provide data for correlating static and dynamic test results.

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APPENDIX A. CTLGROUP REPORT

CTLGroup

July 23, 2020

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Laboratory Evaluation of Dowel Bar Test Methods
CTLGroup Project No. 052167

Dear Dr. Snyder:

As requested, Construction Technology Laboratories, Inc. d/b/a CTLGroup has conducted testing for American Concrete Pavement Association (ACPA) on dowel bars to evaluate the proposed changes to AASHTO T 253[1]. This letter describes the performed scope of work and summarizes the test results and our observations.

BACKGROUND

The current AASHTO test method for evaluating load transfer of dowel bars (Section 5 of AASHTO T 253), involves a uniformly distributed load across a concrete member with a single dowel bar on either side transferring the load to the end support blocks of concrete. The proposed modified version of the test utilizes a larger test specimen that is wide enough to accommodate 4 dowel bars in two joints and applies loading at one edge dowel location over a 12-inch diameter plate while measuring the displacement at the joints. See Figure 1 through Figure 3 for drawings of specimens and test setup as provided by ACPA.

An additional modification to the test method, for both samples, was implemented as follows: (1) apply the standard load, (2) hold the load for 10 minutes, (3) apply additional load to a magnitude 50% greater than the standard load, and (4) then hold the additional load for 10 minutes.

This involved loading to 4,000 and 6,000 pounds for the standard specimen and 9,000 and 13,500 lbs for the modified specimen. The method specifies a load rate of 2,000 pounds per minute for the standard specimen. This rate was used for both the standard load (4,000 pounds) and the overload conditions (6,000 pounds). The load rate used on the modified sample was set to achieve the target loads (9,000 and 13,500 pounds) in the same amount of time as for the standard specimen; resulting in a load rate of 4,500 pounds per minute.

[1] AASHTO T 253-02, *Standard Method of Test for Coated Dowel Bars*, American Association of State Highway and Transportaiton Officials, Washington, D.C., 2002.

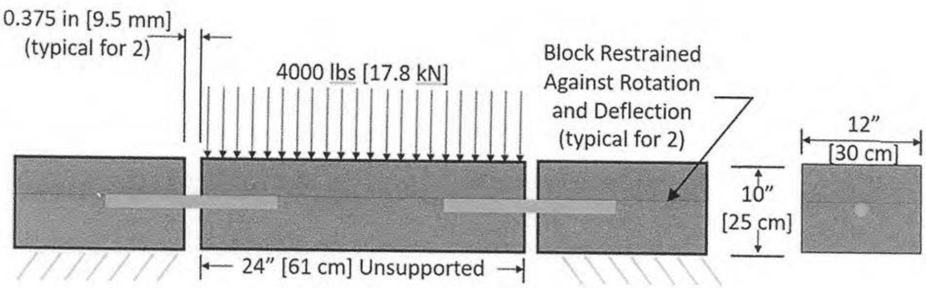

Figure 1. Standard AASHTO T253 Specimen and Test (As Provided by ACPA)

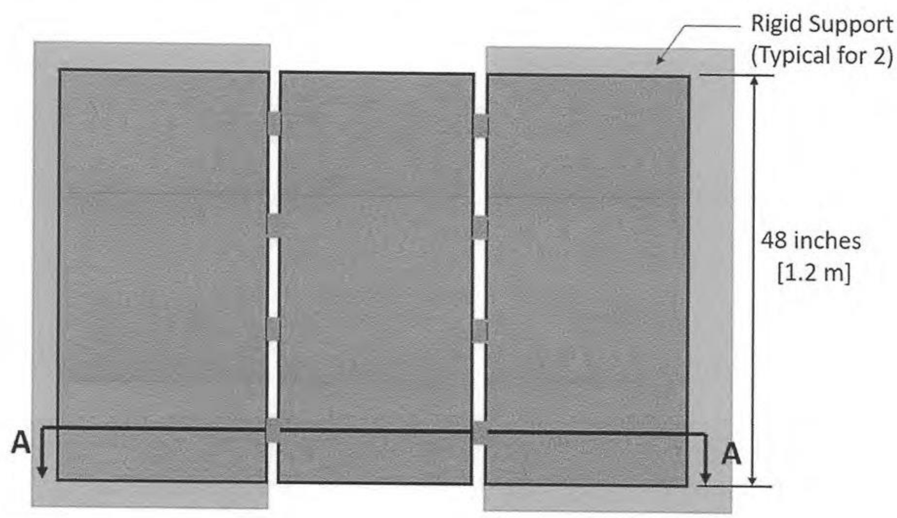

Figure 2. Modified Test Specimen – Plan View (As Provided by ACPA)

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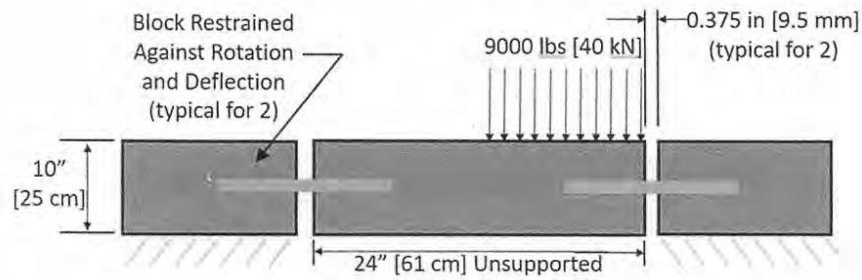

Figure 3. Modified Test Specimen and Test Setup – Section View (As Provided by ACPA)

SPECIMEN FABRICATION

CTLGroup constructed molds to accommodate the fabrication of four (4) test specimens according to the current test method and two (2) test specimens meeting the requirements of the proposed modified test method. Molds were fabricated out of standard dimensional lumber and plywood sheets.

EMBEDDED ANCHORS FOR SPECIMEN TRANSPORT

Drop-in embedded concrete anchors were installed in both specimen types to secure the bracing material to the specimens after fabrication so that they could be moved from the fabrication location to the location of testing without causing any slipping of the joints.

The standard-sized specimens had one set of drop-in anchors placed in the center of each block, as shown in Figure 4. The modified test method specimens had anchors installed on each side of the test specimen, and three centered anchors installed in the top face of the specimen, as shown in Figure 5.

Figure 4. Standard Sample Forms (Red Arrows Indicating the Drop-In Anchors)

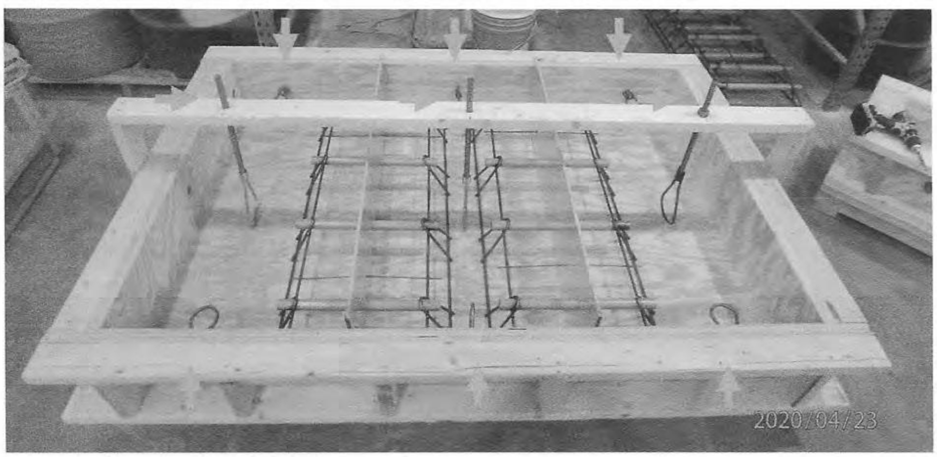

Figure 5. Modified Sample Forms (Red Arrows Indicating the Drop-In Anchors)

After the completion of a curing period of 7 days under wet burlap and plastic sheeting and before moving the specimens to the test location, plywood and steel members were secured to the specimens using the drop-in anchors to prevent any movement in the joints.

DOWEL BAR INSTALLATION

Dowel bars provided by ACPA were used in this testing. The dowel bars were 1.25-inch nominal diameter epoxy-coated bars with a length of 18 inches. The dowel bars used in the standard test method samples were supported by standard rebar chair supports and dimensional lumber to ensure proper placement at the mid-height of the test specimen. The dowel bars used in the modified test method samples were provided by ACPA preinstalled in standard dowel bar chairs. For both sample types, the dowel support structures were secured to the forms to prevent movement during concrete placement, as shown in Figures 4 and 5.

JOINT FORMATION

The joints were formed in each test specimen using plywood covered in aluminum tape. The plywood was run through a wood planer to decrease the thickness such that the final thickness, including the tape, was ⅜ inches. The aluminum air duct tape was used to prevent cement paste bond to the plywood and aid in the removal of the joint forming material after the concrete set. The plywood was fabricated in two horizontal halves. At the mid-depth interfaces of the two plywood pieces, holes were drilled to facilitate the placement of the dowel bars. The interface between the dowel bars and plywood was sealed with vacuum grease to prevent cement paste intrusion. A detail of the joint in the modified sample is shown in Figure 6.

Figure 6. Joint Forming – Modified Test Method Sample

CONCRETE MIXTURE

The concrete was supplied by a local ready-mix company. The mixture was a standard paving mixture used in the Chicago area with 575 lb/yd³ of cement, a water-to-cementitious ratio of 0.42, and a target air content of 6.5%. The aggregates used were a limestone ¾-inch coarse aggregate and natural fine aggregate. The aggregate blend consisted of 42.8% coarse aggregate and 57.2% fine aggregate, by weight (SSD). Compressive strength test specimens were cast on the day of placement and test at 14 and 28 days of age. The strength[2] was 5,590 psi and 6,360 psi at 14 and 28 days, respectively. Young's modulus of elasticity[3] was also measured at 28 days with a result of 4,700,000 psi.

LOAD TESTING

The testing was conducted using CTLGroup's multi-use test apparatus. The test setup for the standard and modified tests are shown in Figure and Figure , respectively. Compressive loads were applied with a 220-kip capacity servo-hydraulic actuator, at the prescribed load rates, and monitored with a 20-kip capacity load cell. Loading was controlled with a closed-loop system that maintained specified loads regardless of specimen displacement. Displacements across the joints were monitored with digital dial indicators. Calibration records for the 20-kip load cell and the dial gages are included as an attachment to this letter.

The outer blocks of both sample sizes were supported on ¾-inch plywood while the middle section of each sample was unsupported, as shown in Figure . The load was transferred to the samples using a rectangular plate covering the entire middle section (minus ½ inch on each end, resulting in 23-inches by 12-inches, to allow for contact between the deflection gages and the concrete) for the standard sized samples, and a 12-inch diameter round plate for the modified method samples. A sheet of rubber (Shore A hardness of 50) was placed between the steel plate and the sample for each test.

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[2] ASTM C39 / C39M-20, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, ASTM International, West Conshohocken, PA, 2020.

[3] ASTM C469 / C469M-14, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*, ASTM International, West Conshohocken, PA, 2014.

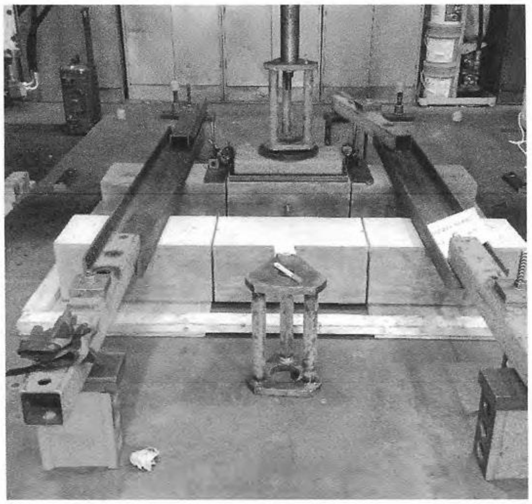


Figure 7. Test Setup for the Standard Test Samples

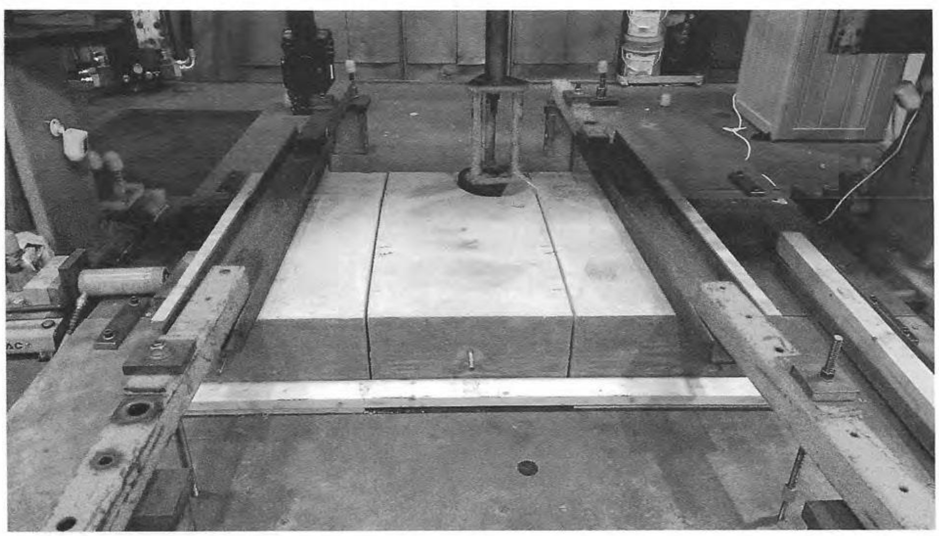


Figure 8. Test Setup for Modified Test Samples

Figure 9. Detail of Support Conditions (Modified Test Sample)

RESULTS

STANDARD TEST METHOD

The standard test specimens were tested with dial gages measuring deflection across each joint. The individual results are included in the attached test reports. Measurements were taken upon initially reaching the specified load of 4,000 lbs or 6,000 lbs, and after 10 minutes of holding the specified load. Readings were also taken 1 minute after unloading. Additionally, Sample 1 was monitored and no change in deflection was observed upon unloading beyond the 1 minute time interval. An overview of the loading procedure is shown in Figure 10.

The average measured deflections are presented in

Table 1. The results indicate that the deflection increases somewhat linearly as the applied load increases. The average measured deflection was 0.0028 and 0.0041 inches for 4,000 and 6,000 lbs load levels, respectively.

The deflection change during the hold period was minimal for both load levels. An increase in deflection of 0.0002 and 0.0003 inches was measured at the 4,000 lb and 6,000 lb load levels, respectively. Approximately 50% of the deflection observed after the first load is applied remained after unloading. The average unloaded deflection value was measured to be 0.0015 inches.

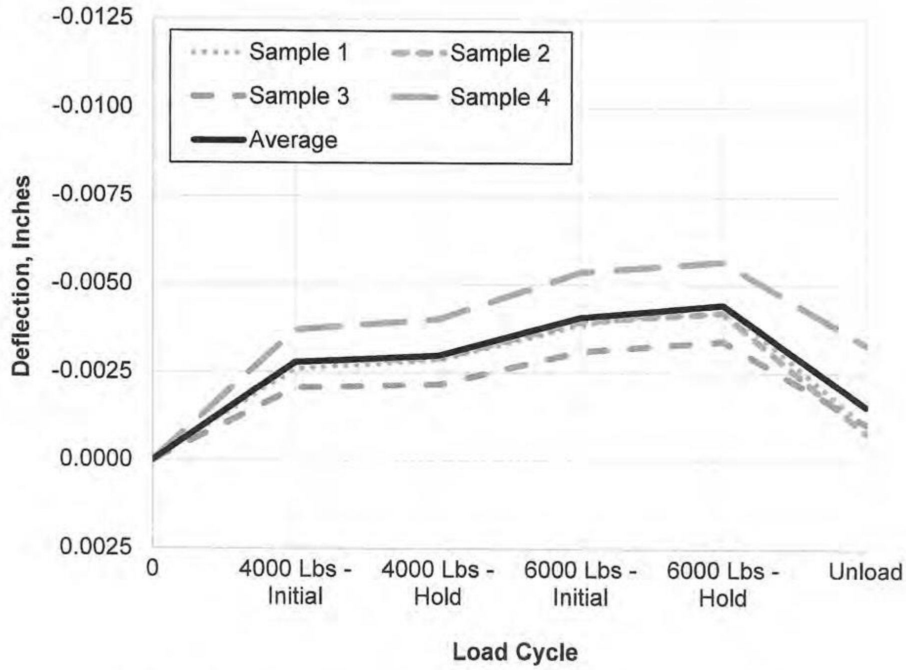


Figure 10. Load and Deflection History for the Standard Sample Test

Table 1 - Standard Sample Test Results

	First load application (4000 lbs)		*Second Load (6000 lbs)*		*Unloaded*
	Average Initial Deflection, Inches	Average Deflection After Hold Period of 10 minutes, Inches	Average Initial Deflection, Inches	Average Deflection After Hold Period of 10 minutes, Inches	Average Deflection 1 Minute After Unloading
Sample 1	-0.0026	-0.0029	-0.0039	-0.0045	-0.0010
Sample 2	-0.0028	-0.0029	-0.0040	-0.0042	-0.0008
Sample 3	-0.0021	-0.0022	-0.0031	-0.0034	-0.0011
Sample 4	-0.0037	-0.0040	-0.0054	-0.0057	-0.0033
Average	**-0.0028**	**-0.0030**	**-0.0041**	**-0.0044**	**-0.0015**

PROPOSED MODIFIED TEST METHOD

The modified test samples were tested at all four corners. One digital dial gage was placed to measure deflection across the joint at the location of loading (*primary deflection*). The position of the second digital dial gage was varied to capture to measure the deflection under three different locations (*secondary deflection*), as shown in Figure 11.

Location 1: Across the closest dowel bar on the non-loaded joint (2 of the 4 corner tests),
Location 2: Across the furthest dowl bar along the loaded joint (1 of 4 corner tests), and
Location 3: The furthest dowel bar along the non-loaded joint (1 of 4 corner tests)

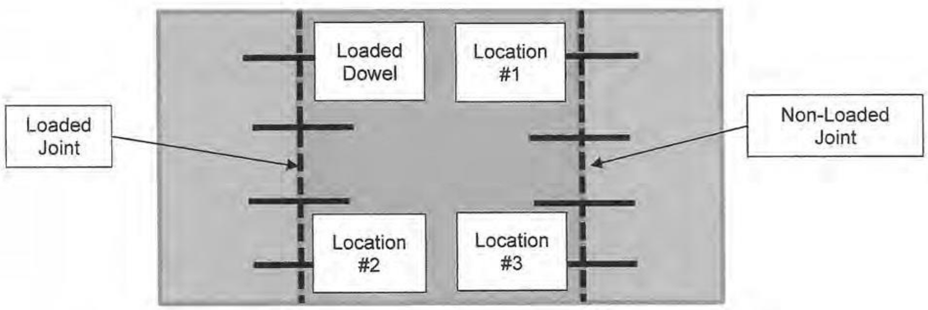


Figure 11. Secondary Deflection Measurement Locations

The testing sequence was reversed between the two tested specimens to evaluate the influence of the test sequence on the results.

Considering the results for deflection at the point of loading, the test results of the first load cycle are similar to the standard sample for both modified sample tests; see Figure and Figure . The subsequent testing resulted in increasing deflections except for the last test in sequence for Sample 2 (secondary deflection point at Location 3). Secondary point deflections are presented in Figure and Figure .

The impact of the hold time at each load level was similar to the standard sample with very little change in deflection for both the gage at the point of loading and the secondary deflection gage. The amount of deflection measured after removing all load resulted in values similar to the standard sample with values ranging from 0.0038 inches deflection to uplift of 0.0005 inches.

For both samples, the loading and deflection measurements at Location 1 above showed the greatest deflection, regardless of the test sequence. The deflection at Location 2 exhibited lower measured deflection values and actually showed slight uplift in sample 2 and minimal deflection in sample 1. All measurements at Location 3 exhibited uplift; both samples and at both load levels.

The observed results indicate the center panel load preferentially transfers to the adjacent panel through the joint rather than along the joint line across the dowels, based on the higher deflections at location 1 rather than location 2. Location 3 data indicates an uplift condition occurs.

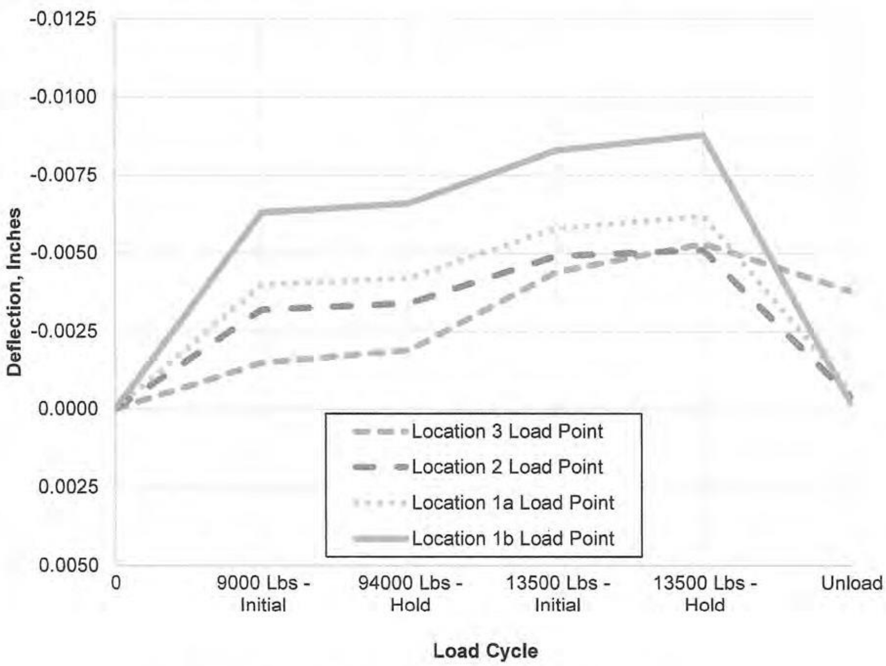


Figure 12. Modified Sample Size, Sample 1 Load Point Data

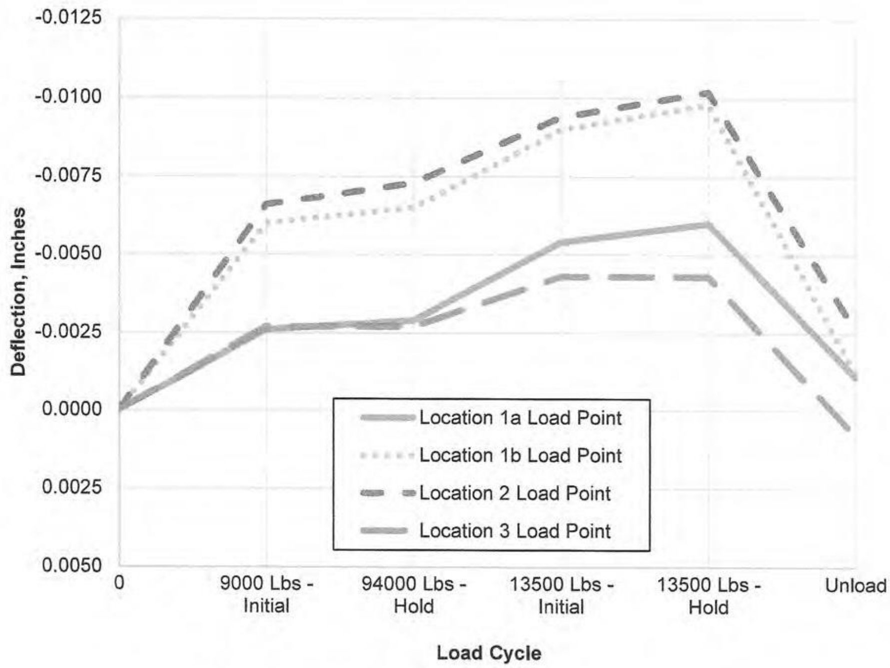


Figure 13. Modified Sample Size, Sample 2 Load Point Data

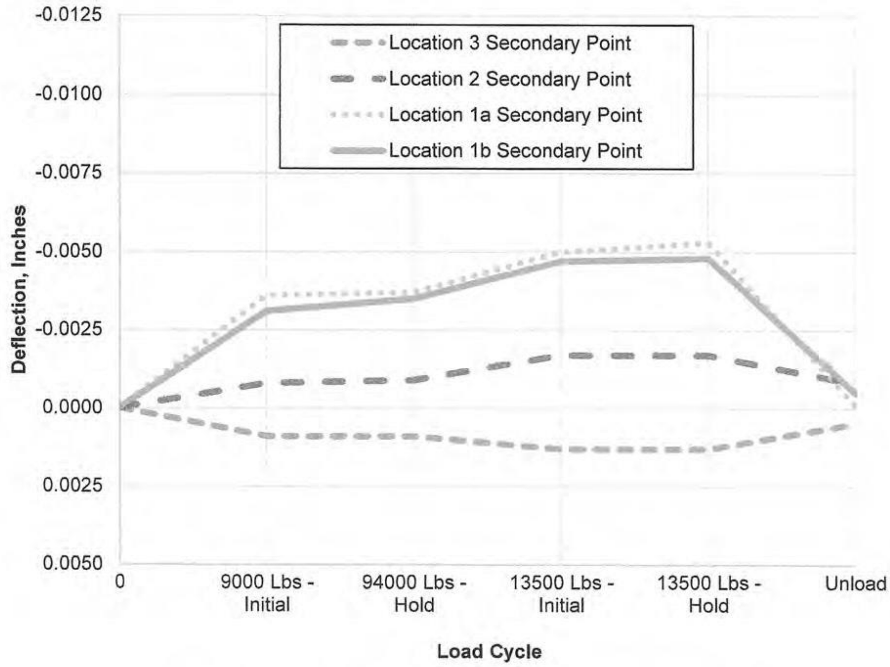


Figure 14. Modified Sample Size, Sample 1 Secondary Point Data

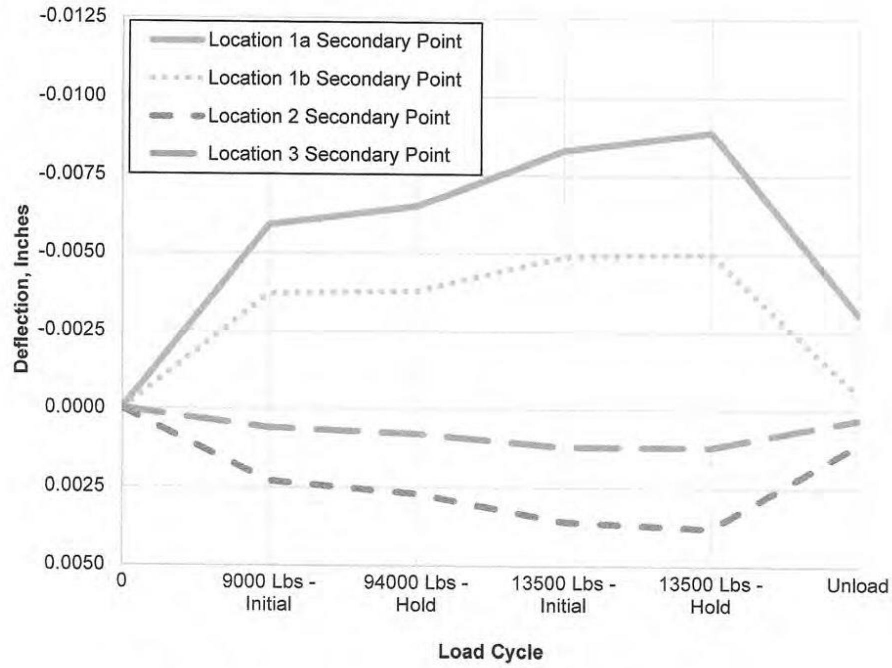


Figure 15. Modified Sample Size, Sample 2 Secondary Point Data

Table 2 - Modified Sample Results

		First load application (9000 lbs)				*Second Load (13500 lbs)*				*Unloaded*	
		Average Initial Deflection, Inches		Average Deflection After Hold Period of 10 minutes, Inches		Average Initial Deflection, Inches		Average Deflection After Hold Period of 10 minutes, Inches		Average Deflection 1 Minute After Unloading	
Sample	Secondary Deflection Measurement Points (In Sequence)	Load Point	2nd Point	Load Point	2nd Point	Load Point	2nd Point	Load Point	2nd Point	Load Point	2nd Point
Sample 1	Location 3	-0.0015	0.0009	-0.0019	0.0009	-0.0044	0.0013	-0.0053	0.0013	-0.0038	0.0005
	Location 2	-0.0032	-0.0008	-0.0034	-0.0009	-0.0049	-0.0017	-0.0051	-0.0017	-0.0004	-0.0008
	Location 1a	-0.0040	-0.0036	-0.0042	-0.0037	-0.0058	-0.0050	-0.0062	-0.0053	-0.0006	-0.0001
	Location 1b	-0.0063	-0.0031	-0.0066	-0.0035	-0.0083	-0.0047	-0.0088	-0.0048	-0.0002	-0.0005
Sample 2	Location 1a	-0.0026	-0.0059	-0.0029	-0.0065	-0.0054	-0.0083	-0.0060	-0.0089	-0.0011	-0.0031
	Location 1b	-0.0060	-0.0037	-0.0065	-0.0038	-0.0090	-0.0049	-0.0098	-0.0050	-0.0012	-0.0005
	Location 2	-0.0066	0.0023	-0.0073	0.0027	-0.0094	0.0036	-0.0102	0.0038	-0.0026	0.0011
	Location 3	-0.0027	0.0006	-0.0027	0.0008	-0.0043	0.0012	-0.0043	0.0012	0.0008	0.0003

SUMMARY

The described testing program consisted of mold and specimen fabrication, and testing according to the standard AASHTO T 253 load-deflection test procedure with an additional load step, and a proposed modified sample and loading configuration of this test method.

The modified sample test procedure resulted in the concentration of the load one dowel bar and also provides information as to the degree of load distribution across a panel both along a joint and to the far joint by measuring deflection at several locations. The project also identified practical approaches to ensure sample integrity while moving the samples in a laboratory setting through the implementation of drop-in anchors. The results indicate the initial loading for the modified sample results in similar deflection values at the point of loading to the results of the standard sample. Subsequent loadings tend to increase the measured deflection values. Dwell times at each load intensity had little impact on the measured deflection values.

We appreciate the opportunity to work with you on this project. Please let us know if you have any questions or concerns.

Sincerely,

CONSTRUCTION TECHNOLOGY LABORATORIES, INC., dba CTLGroup

Ben Birch, PE (IL, CO, VT, TX)
Concrete and Cement-Based Materials
BBirch@CTLGroup.com
Phone: (847) 972-3246

Attachments:
1. *Data Sheets*
2. *Calibration Records*
3. *C39/C469 Test Results*

CTLGROUP
www.CTLGroup.com

APPENDIX B. CTLGROUP RAW DATA

Loading At Corner 1-1 | **PROJECT NO.: 052167**

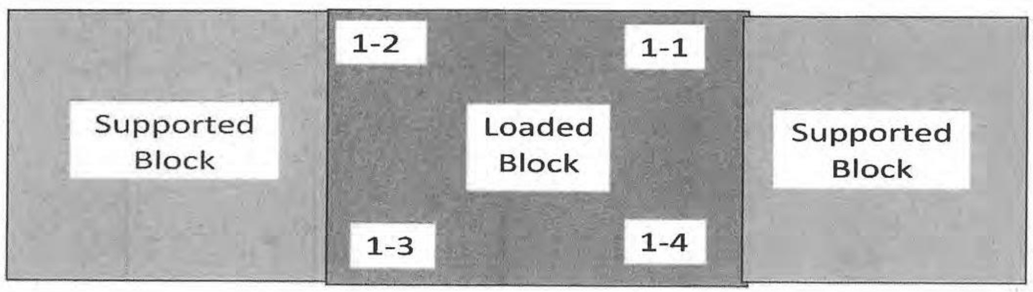
5|20|20

Weight of Test Fixture:	63	#
Phase 1 Load (Total):	9000	#
Phase 2 Load (Total):	13500	#
ZERO DIAL GAGES		in.
Initial Deflection (T=0):	0	in.

INITIAL GAP = 3/8"

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (1-1):	-.0015	in.
Measured Deflection (1-3):	+0.0009	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (1-1):	-0.0019	in.
Measured Deflection (1-3):	+0.0009	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (1-1):	-0.0044	in.
Measured Deflection (1-3):	+0.0013	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (1-1):	-0.0053	in.
Measured Deflection (1-3):	+0.0013	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (1-1):	-0.0038	in.
Measured Deflection (1-3):	+0.0005	in.



Loading At Corner 1-2

PROJECT NO.: 052167

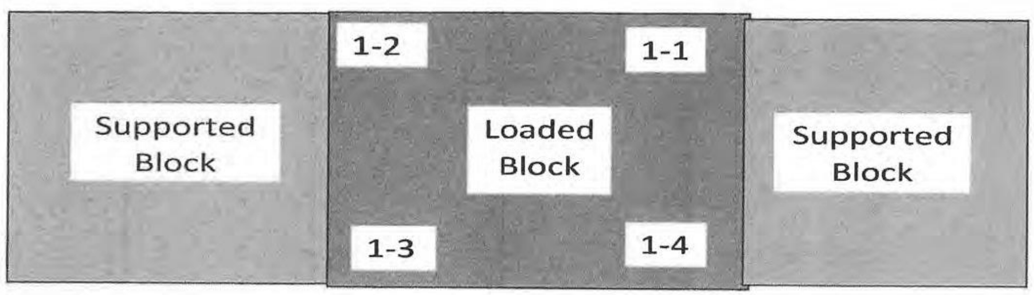
Weight of Test Fixture:	63 #
Phase 1 Load (Total):	9000 #
Phase 2 Load (Total):	13500 #
ZERO DIAL GAGES	in.
Initial Deflection (T=0):	0 in.

INITIAL GAP = 3/8"

5/20/20 29:44 AM

Phase 1			
Ramp Rate:	4500	#/min.	
Load Applied:	8937	#	
At Phase 1 Load:			
Measured Deflection (1-2):	-0.0032	in.	
Measured Deflection (1-3):	-0.0008	in.	
At T12 min (+10min after Phase 1 Load):			
Measured Deflection (1-2):	~~-0.0032~~	in.	-0.0034
Measured Deflection (1-3):	~~-0.0008~~	in.	-0.0009

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (1-2):	-0.0049	in.
Measured Deflection (1-3):	-0.0017	in.
At T23 min (+10min after Phase 2 Load):	~~-0.00~~	
Measured Deflection (1-2):	-0.0051	in.
Measured Deflection (1-3):	-0.0017	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (1-2):	-0.0004	in.
Measured Deflection (1-3):	-0.0008	in.



Loading At Corner 1-3

PROJECT NO.: 052167

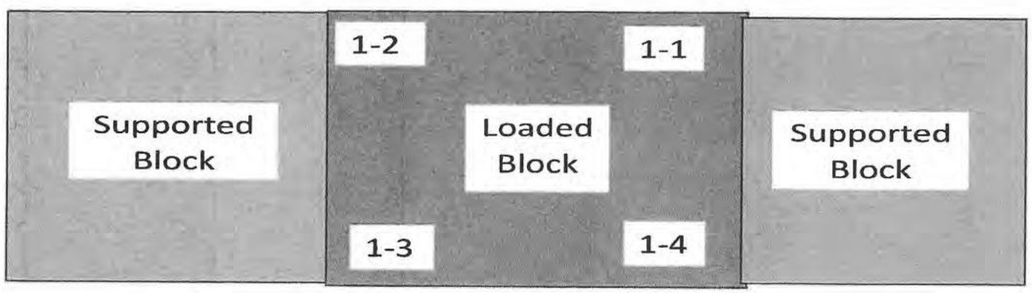
5/20/20 @ 10:44 AM

Weight of Test Fixture:	63 #
Phase 1 Load (Total):	9000 #
Phase 2 Load (Total):	13500 #
ZERO DIAL GAGES	in.
Initial Deflection (T=0):	0 in.

INITIAL GAP = 3/8"

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (1-3):	-0.0040	in.
Measured Deflection (1-4):	-0.0036	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (1-3):	-0.0042	in.
Measured Deflection (1-4):	-0.0037	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (1-3):	-0.0058	in.
Measured Deflection (1-4):	-0.0050	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (1-3):	-0.0062	in.
Measured Deflection (1-4):	-0.0053	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (1-3):	-0.0006	in.
Measured Deflection (1-4):	-0.0001	in.



Loading At Corner 1-4

PROJECT NO.: 052167

5/20/20 @ 11:25 AM

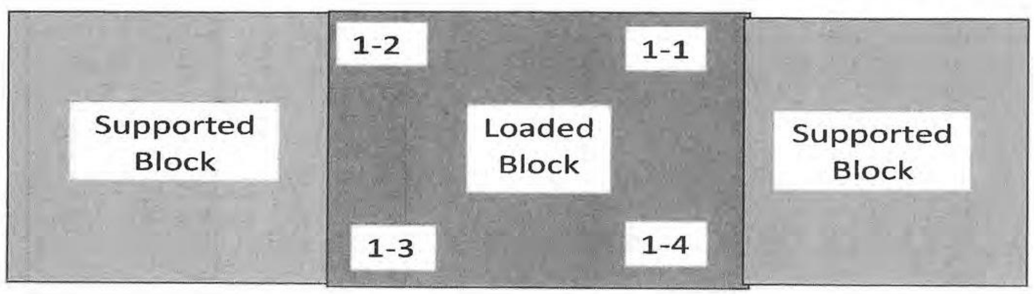
Weight of Test Fixture:	63	#
Phase 1 Load (Total):	9000	#
Phase 2 Load (Total):	13500	#
ZERO DIAL GAGES		in.
Initial Deflection (T=0):	0	in.

INITIAL GAP = 3/8"

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (1-4):	-0.0063	in.
Measured Deflection (1-3):	-0.0031	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (1-4):	-0.0066	in.
Measured Deflection (1-3):	-0.0035	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (1-4):	-0.0083	in.
Measured Deflection (1-3):	-0.0047	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (1-4):	-0.0088	in.
Measured Deflection (1-3):	-0.0048	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (1-4):	-0.0202*	in.
Measured Deflection (1-3):	-0.0005	in.

* TRANSDUCER MOVED ON RELEASE OF LOAD



Loading At Corner 2-1

PROJECT NO.: 052167

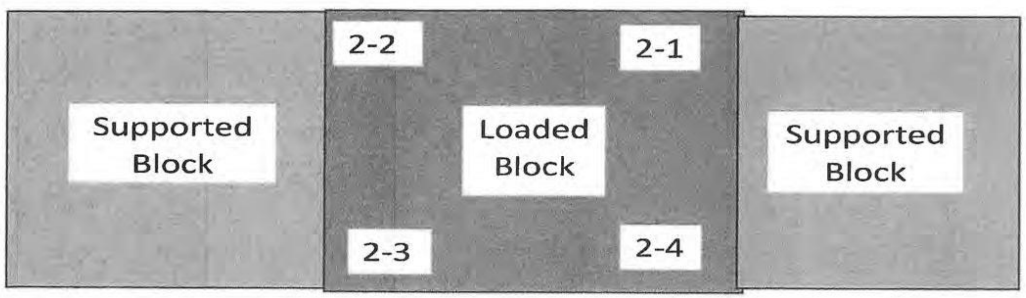
5/20/20 @ 2:13 PM

Weight of Test Fixture:	63	#
Phase 1 Load (Total):	9000	#
Phase 2 Load (Total):	13500	#
ZERO DIAL GAGES		in.
Initial Deflection (T=0):	0	in.

INITIAL GAP = 3/8"

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (2-1):	-0.0026	in.
Measured Deflection (2-3):	-0.0059	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (2-1):	-0.0029	in.
Measured Deflection (2-3):	-0.0065	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (2-1):	-0.0054	in.
Measured Deflection (2-3):	-0.0083	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (2-1):	-0.0060	in.
Measured Deflection (2-2):	-0.0089	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (2-1):	-0.0011	in.
Measured Deflection (2-2):	-0.0031	in.



Loading At Corner 2-2

PROJECT NO.: 052167

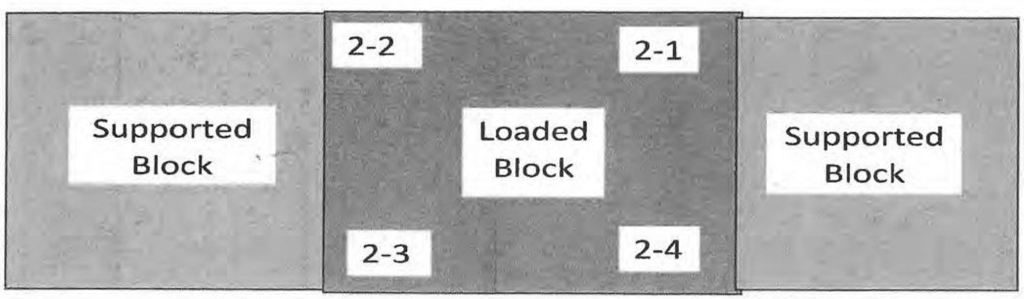
Weight of Test Fixture: 63 #
Phase 1 Load (Total): 9000 #
Phase 2 Load (Total): 13500 #

ZERO DIAL GAGES in.
Initial Deflection (T=0): 0 in.

INITIAL GAP: 3/8" 5/20/20 @ 2:53 PM

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (2-2):	-0.0060	in.
Measured Deflection (2-3):	-0.0037	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (2-2):	-0.0065	in.
Measured Deflection (2-3):	-0.0038	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (2-2):	-0.0090	in.
Measured Deflection (2-3):	-0.0049	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (2-2):	-0.0098	in.
Measured Deflection (2-3):	-0.0050	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (2-2):	-0.0012	in.
Measured Deflection (2-3):	-0.0005	in.



Loading At Corner 2-3

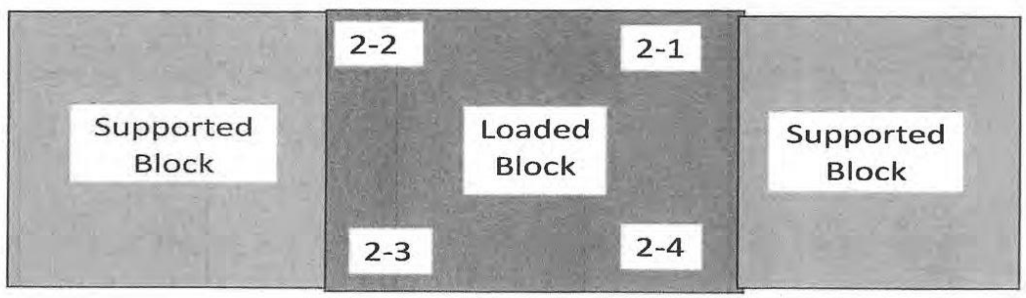
PROJECT NO.: 052167

Weight of Test Fixture:	63	#
Phase 1 Load (Total):	9000	#
Phase 2 Load (Total):	13500	#
ZERO DIAL GAGES		in.
Initial Deflection (T=0):	0	in.

INITIAL GAP = 3/8"   5/20/20 @ 3:31 PM

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (2-3):	-0.0066	in.
Measured Deflection (2-2):	+0.0023	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (2-3):	-0.0073	in.
Measured Deflection (2-2):	+0.0027	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (2-3):	-0.0094	in.
Measured Deflection (2-2):	+0.0036	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (2-3):	-0.0102	in.
Measured Deflection (2-2):	+0.0038	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (2-3):	-0.0026	in.
Measured Deflection (2-2):	+0.0011	in.



Loading At Corner 2-4

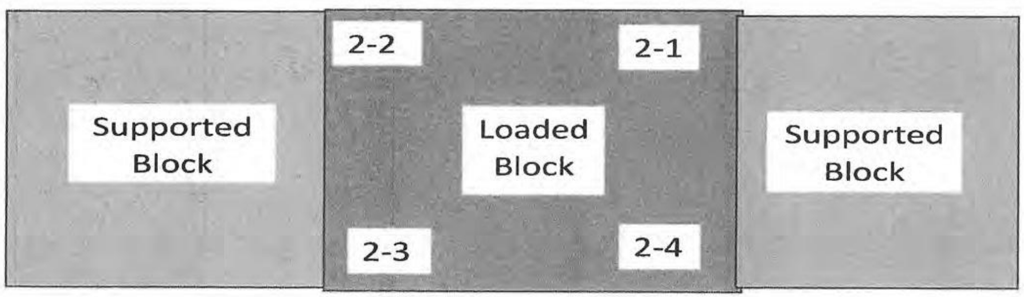
PROJECT NO.: 052167

Weight of Test Fixture:	63 #
Phase 1 Load (Total):	9000 #
Phase 2 Load (Total):	13500 #
ZERO DIAL GAGES	in.
Initial Deflection (T=0):	0 in.

INITIAL GAP= 3/8"   5/20/20 @ 4:09 PM

Phase 1		
Ramp Rate:	4500	#/min.
Load Applied:	8937	#
At Phase 1 Load:		
Measured Deflection (2-4):	-0.0027	in.
Measured Deflection (2-3):	+0.0006	in.
At T12 min (+10min after Phase 1 Load):		
Measured Deflection (2-4):	-0.0027	in.
Measured Deflection (2-3):	+0.0008	in.

Phase 2:		
Ramp Rate:	4500	#/min.
Load Applied:	13437	#
At Phase 2 Load:		
Measured Deflection (2-4):	-0.0043	in.
Measured Deflection (2-3):	+0.0012	in.
At T23 min (+10min after Phase 2 Load):		
Measured Deflection (2-4):	-0.0043	in.
Measured Deflection (2-3):	+0.0012	in.
Remove Load		
At T24 (1min after unload):		
Measured Deflection (2-4):	+0.0008	in.
Measured Deflection (2-3):	+0.0003	in.



Loading Small Slab #1

PROJECT NO.: 052167

5/21/20

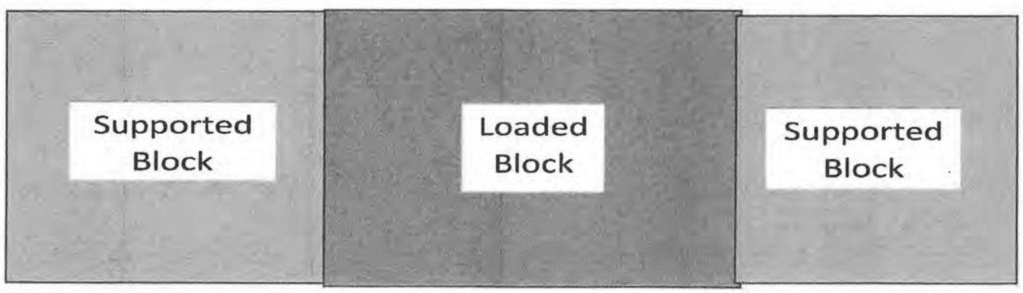
Weight of Test Fixture:	134 #	+32#
Phase 1 Load (Total):	4000 #	
Phase 2 Load (Total):	6000 #	
ZERO DIAL GAGES	in.	
Initial Deflection (T=0):	0 in.	

INITIAL GAP = 3/8"

Phase 1		
Ramp Rate:	2000	#/min.
Load Applied:	3866	# -32#
At Phase 1 Load:		
Displacement - Side 1:	-0.0014	in.
Displacement - Side 2:	-0.0038	in.
At T12 min (+10min after Phase 1 Load):		
Displacement - Side 1:	-0.0017	in.
Displacement - Side 2:	-0.0040	in.

	S1	S2
24h :	-0.0012	-0.0039
26h :	-0.0020	-0.0048
20h :	-0.0002	-0.0004

Phase 2:		
Ramp Rate:	2000	#/min.
Load Applied:	5866	# -32#
At Phase 2 Load:		
Displacement - Side 1:	-0.0027	in.
Displacement - Side 2:	-0.0050	in.
At T23 min (+10min after Phase 2 Load):		
Displacement - Side 1:	-0.0033	in.
Displacement - Side 2:	-0.0056	in.
Remove Load		
At T24 (1min after unload):		
Displacement - Side 1:	-0.0010	in.
Displacement - Side 2:	-0.0010	in.



Loading Small Slab #2 PROJECT NO.: 052167

Weight of Test Fixture: 134 # +32#

Phase 1 Load (Total): 4000 #

Phase 2 Load (Total): 6000 #

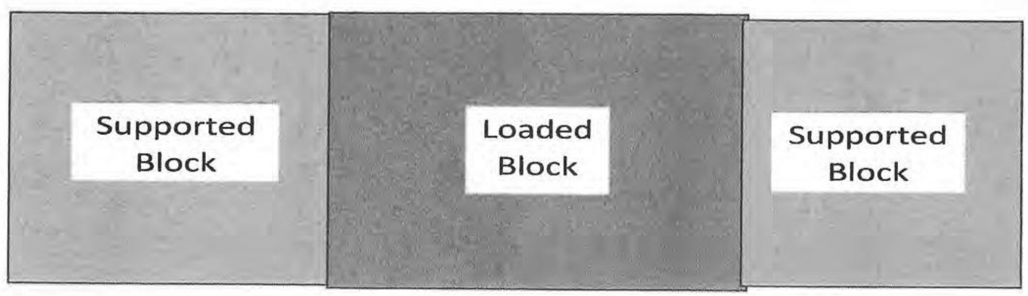
ZERO DIAL GAGES in. INITIAL GAP = 3/8" 5/21/20

Initial Deflection (T=0): 0 in.

Phase 1		
Ramp Rate:	2000	#/min.
Load Applied:	3866	# - 32#
At Phase 1 Load:		
Displacement - Side 1:	-0.0029	in.
Displacement - Side 2:	-0.0027	in.
At T12 min (+10min after Phase 1 Load):		
Displacement - Side 1:	-0.0030	in.
Displacement - Side 2:	-0.0028	in.

	S1	S2
@ 4k :	-0.0025	-0.0024
@ 6k :	-0.0034	-0.0033
@ 0k ;	-0.0001	-0.0001

Phase 2:		
Ramp Rate:	2000	#/min.
Load Applied:	5866	# - 32#
At Phase 2 Load:		
Displacement - Side 1:	-0.0041	in.
Displacement - Side 2:	-0.0038	in.
At T23 min (+10min after Phase 2 Load):	~~-0.0044~~	
Displacement - Side 1:	~~-0.0040~~	~~in.~~ -0.0044
Displacement - Side 2:		in. -0.0040
Remove Load		
At T24 (1min after unload):		
Displacement - Side 1:	-0.0009	in.
Displacement - Side 2:	-0.0007	in.



Loading Small Slab #3 **PROJECT NO.: 052167**

Weight of Test Fixture:	134 # + 32 #
Phase 1 Load (Total):	4000 #
Phase 2 Load (Total):	6000 #
ZERO DIAL GAGES	in.
Initial Deflection (T=0):	0 in.

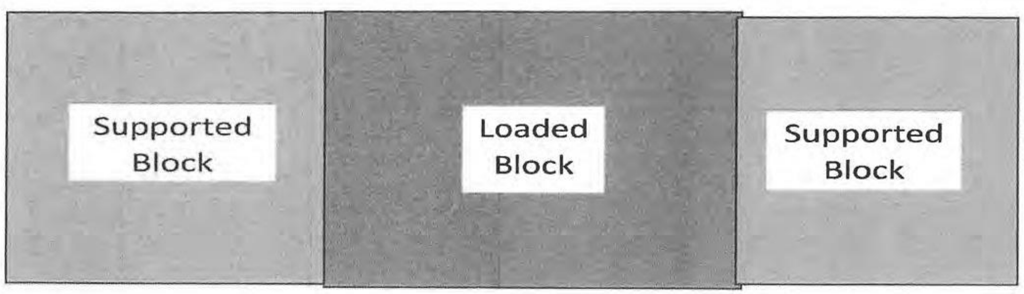
INITIAL GAP = 3/8

5/21/20

Phase 1		
Ramp Rate:	2000	#/min.
Load Applied:	3866	# - 32#
At Phase 1 Load:		
Displacement - Side 1:	-0.0021	in.
Displacement - Side 2:	-0.0020	in.
At T12 min (+10min after Phase 1 Load):		
Displacement - Side 1:	-0.0022	in.
Displacement - Side 2:	-0.0021	in.

	S1	S2
24hr	-0.0017	-0.0017
26hr	-0.0025	-0.0022
20hr	-0.0002	-0.0000

Phase 2:		
Ramp Rate:	2000	#/min.
Load Applied:	5866	# - 32#
At Phase 2 Load:		
Displacement - Side 1:	-0.0031	in.
Displacement - Side 2:	-0.0031	in.
At T23 min (+10min after Phase 2 Load):		
Displacement - Side 1:	-0.0034	in.
Displacement - Side 2:	-0.0034	in.
Remove Load		
At T24 (1min after unload):		
Displacement - Side 1:	-0.0009	in.
Displacement - Side 2:	-0.0012	in.



Loading Small Slab #4

PROJECT NO.: 052167

5/21/20

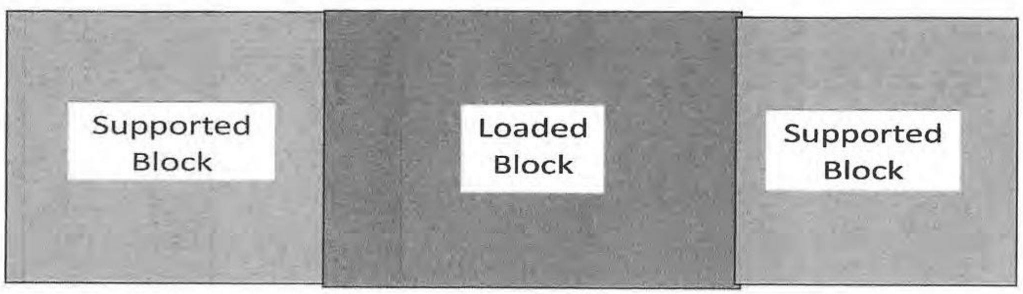
Weight of Test Fixture:	134 #	+32 #
Phase 1 Load (Total):	4000 #	
Phase 2 Load (Total):	6000 #	
ZERO DIAL GAGES	in.	
Initial Deflection (T=0):	0 in.	

INITIAL GAP = 3/8

Phase 1		
Ramp Rate:	2000	#/min.
Load Applied:	3866	# - 32 #
At Phase 1 Load:		
Displacement - Side 1:	-0.0028	in.
Displacement - Side 2:	-0.0046	in.
At T12 min (+10min after Phase 1 Load):		
Displacement - Side 1:	-0.0030	in.
Displacement - Side 2:	-0.0050	in.

	S1	S2
24hr:	-0.0023	-0.0006
26hr:	-0.0033	-0.0019
20hr:	-0.0002	-0.0002

Phase 2:		
Ramp Rate:	2000	#/min.
Load Applied:	5866	# - 32 #
At Phase 2 Load:		
Displacement - Side 1:	-0.0042	in.
Displacement - Side 2:	-0.0065	in.
At T23 min (+10min after Phase 2 Load):		
Displacement - Side 1:	-0.0044	in.
Displacement - Side 2:	-0.0069	in.
Remove Load		
At T24 (1min after unload):		
Displacement - Side 1:	-0.0012	in.
Displacement - Side 2:	-0.0054	in.



ASSURANCE Technologies, Inc.

MAIN OFFICE / SHOWROOM / LAB
1251 Humbracht Circle, Unit A
Bartlett, IL 60103
Phone: (630) 550-5000 - Fax: (630) 550-5001

SALES OFFICES
Rockford, IL (815) 963-1971
Milwaukee, WI (414) 481-9606
Web Site: www.ATIQuality.com

ISO / IEC 17025:2005 ACCREDITED

PJLA Calibration
Accreditation # 59361

Certificate No: 193522497-RC-20200114

Certificate of Calibration
Indicator

Customer Name and Address:
CTL Group
5400 Old Orchard Road
Skokie IL 60077

Calibration Date:	1/14/2020 Rev. 00
Interval:	12 months
Calibration Due:	1/14/2021
Procedures:	DP-MET217, ASME B891.10M

Gage ID #:	193522497
Serial No.:	193522497
Manufacturer:	CDI
Model:	Q2110
Range/Size:	1
Type:	Digital
Location:	ATI Onsite

Units of Measure:	in
Temperature:	71.4 °F **Humidity:** 37.8 %RH
Resolution:	0.0001
Calibrated By:	Robert Corcoran

AS FOUND / AS LEFT READINGS

Test Points	Nominal	Minimum	Maximum	Actual	Deviation	Uncertainty	Compliance w/Spec
1	0.1000	0.0999	0.1001	0.1000	0.0000	0.000080	Pass
2	0.2500	0.2499	0.2501	0.2500	0.0000	0.000081	Pass
3	0.5000	0.4999	0.5001	0.5000	0.0000	0.000083	Pass
4	0.7500	0.7499	0.7501	0.7500	0.0000	0.000084	Pass
5	1.0000	0.9999	1.0001	1.0000	0.0000	0.000086	Pass
Repeatability	0.0000	-0.0001	0.0001	0.0000	0.0000	0.000079	Pass

Comments Below:

Note: Pass* = Pass Within Uncertainty

Certification Statement:

This calibration was performed in accordance with requirements of ISO/IEC 17025:2005 with measuring standards traceable to SI Units through NIST. The uncertainties reported are for the confidence probability of not less than 95% with a coverage factor of K=2. Uncertainties were taken into account in determining pass/fail status. All results within this certificate relate only to the item(s) calibrated. Testing was completed per above referenced procedures/standards.

Standards Used	Gage ID #	Cal. Due Date	ATI Traceability #
Square Gage Block Set	G-3007	02/19/2020	G-3007-AG-2019/02/19
Thermohygrometer	G-1128	06/27/2020	G-1128-MS-20190627

Approved By: [signature]

Metrologist #: 420

ASSURANCE Technologies, Inc.

MAIN OFFICE / SHOWROOM / LAB
1251 Humbracht Circle, Unit A
Bartlett, IL 60103
Phone: (630) 550-5000 - Fax: (630) 550-5001

SALES OFFICES
Rockford, IL (815) 963-1971
Milwaukee, WI (414) 481-9606
Web Site:www.ATIQuality.com

ISO / IEC 17025:2005 ACCREDITED

ilac-MRA

PJLA Calibration
Accreditation # 59361

Certificate No: 152735845-RC-20200420

Certificate of Calibration
Indicator

Customer Name and Address:
CTL Group
5400 Old Orchard Road
Skokie IL 60077

Calibration Date: 4/20/2020 Rev. 00
Interval: 12 months
Calibration Due: 4/20/2021
Procedures: DP-MET217, ASME B891.10M

Gage ID #: 152735845
Serial No.: 152735845
Manufacturer: CDI
Model: Q2110
Range/Size: 1
Type: Digital
Location: ATI Onsite

Units of Measure: in
Temperature: 71.8 °F **Humidity:** 24.2 %RH
Resolution: 0.0001
Calibrated By: Robert Corcoran

AS FOUND / AS LEFT READINGS

Test Points	Nominal	Minimum	Maximum	Actual	Deviation	Uncertainty	Compliance w/Spec
1	0.1000	0.0995	0.1005	0.1000	0.0000	0.000080	Pass
2	0.2500	0.2495	0.2505	0.2500	0.0000	0.000081	Pass
3	0.5000	0.4995	0.5005	0.5000	0.0000	0.000083	Pass
4	0.7500	0.7495	0.7505	0.7500	0.0000	0.000084	Pass
5	1.0000	0.9995	1.0005	1.0000	0.0000	0.000086	Pass
Repeatability	0.0000	-0.0005	0.0005	0.0000	0.0000	0.000079	Pass

Comments Below:

Note: Pass* = Pass Within Uncertainty

Certification Statement:

This calibration was performed in accordance with requirements of ISO/IEC 17025:2005 with measuring standards traceable to SI Units through NIST. The uncertainties reported are for the confidence probability of not less than 95% with a coverage factor of K=2. Uncertainties were taken into account in determining pass/fail status. All results within this certificate relate only to the item(s) calibrated. Testing was completed per above referenced procedures/standards.

Standards Used	Gage ID #	Cal. Due Date	ATI Traceability #
Gage Block Set/Bore Gage Calibrator	G-8008	07/31/2020	G-8008-SH-2019/07/31
Thermohygrometer	G-S7-G008	06/27/2020	G-1128-MS-20190627

Approved By: [signature]

Metrologist #: 420

Form# ATI-566 Rev 018 12/08/17

Page 1 of 1

CTLGROUP

Instrument Calibration Report

Corporate Office and Laboratory: 5400 Old Orchard Road, Skokie, IL 60077-1030

Calibration Report Number: 25-05-11-2020-1

Instrument Under Calibration

Manufacturer: Interface
Model Number: 1020ACK-25K
Serial Number: 526693A
Full Scale (kibf): -15

Signal Conditioner

Manufacturer: MTS
Model Number: 494.16
Serial Number: 0009121935

N.I.S.T. Traceable Calibration References Utilized

Load Cell
Manufacturer: Interface
Model Number: 1620AJH-25K
Serial Number: 367990
Calibration Due Date: 2/18/2021
Uncertainty (kibf): 0.00068
Class A Range (kibf): 0.27279
Calibration Report No: 367990-20200218

Read Out
Manufacturer: Interface
Model Number: 9840-200-1
Serial Number: 20136
Calibration Due Date: 2/24/2021
Uncertianty (kibf): 0.000916623
Calibration Report No: 20136-20

Readout Device

Manufacturer: MTS
Model Number: Flextest100
Serial Number: 09119554_H
Channel: S5-J1A
Resolution (kibf): 0.001

Calibration Information

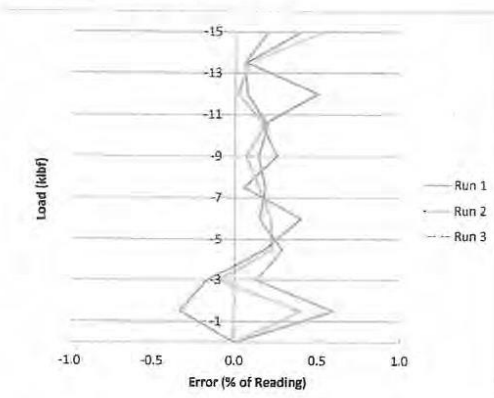
Temperature (°F): 75.8
Relative Humidity: 49.8%
System Condition: Good
Calibration Method: Set The Force
Procedure Used: QOW 25-001

MTE Utilized

Manufacturer	Model Number	Serial Number	Calibration Due Date
Omega	HH314	80401789	9/6/2020

Compression Calibration Data				Error			
				Relative Error		Fixed Error	
Reference (kibf)	Run 1 (kibf)	Run 2 (kibf)	Run 3 (kibf)	Max (%)	Repeatability (%)	Max (kibf)	Repeatability (kibf)
0.00	0.000	0.000	0.000	0.00	0.00	0.00	0.00
-1.50	-1.495	-1.509	-1.506	0.60	0.93	-0.01	0.01
-3.00	-2.995	-3.004	-2.997	-0.17	0.30	0.00	0.01
-4.50	-4.508	-4.513	-4.510	0.29	0.11	-0.01	0.00
-6.00	-6.024	-6.009	-6.013	0.40	0.25	-0.02	0.01
-7.50	-7.504	-7.514	-7.511	0.19	0.13	-0.01	0.01
-9.00	-9.023	-9.013	-9.006	0.26	0.19	-0.02	0.02
-10.50	-10.517	-10.521	-10.519	0.20	0.04	-0.02	0.00
-12.00	-12.060	-12.009	-12.002	0.50	0.48	-0.06	0.06
-13.50	-13.509	-13.508	-13.511	0.08	0.02	-0.01	0.00
-15.00	-15.060	-15.030	-15.080	0.53	0.33	-0.08	0.05
0.00	0.000	0.000	0.000	0.00	0.00	0.00	0.00

Relative error at zero is expressed as percent of full scale



Calibration Results

As Found: N/A
As Left: In Tolerance
Expanded Uncertainty (kibf): 0.063

Relative Error
Max (%): 0.60
Repeatability (%): 0.93
Return to Zero (%): 0.00

Fixed Error
Max (kibf): 0.08
Repeatability (kibf): 0.06
Return to Zero:(kibf) 0.00

Shunt Calibration

Shunt Value (Ω)	Reading (kibf)
80.6k	12.906

Signal Conditioner Settings
Ssensor Calibration File: SN 526693A.scf
Fullscale Min: -15
Fullscale Max: 15
Units: kips
Polarity: Normal
Pre-amp: 540.36
Post-amp: 1.48434
Excitation(p-p): 10
DeltaK: 0.9888

This calibration has been performed using procedures described by the manufacturer, CTLGroup, or both. Results of this calibration apply only to the items described herein. Certificates of calibration for standards used are on file. This report may not be reproduced in any format unless the reproduction is a complete and true copy of the original. Calibrations are performed with standards whose values and measurements are traceable to the National Institute of Standards and Technology.

Calibration Date: 5/11/2020
Calibration Due Date: 5/11/2021

Performed By: Muro **Signature:** ______ **Date:** 6/4/2020

Checked By: Loeppert **Signature:** ______ **Date:** 6/15/2020

Quality: **Signature:** ______ **Date:** 6/22/2020

Page 1 of 1

CTLGROUP

Client:	M. Snycer / ACPA	CTLGroup Project No: **052167**
Project:	**Evaluation of Dowel Bar Test Methods**	CTLGroup Project Mgr.: **B. Birch**
Contact:	**M. Snyder**	Technician: **W. Demharter**
Date Reported:	**May 7, 2020**	Approved: **J. Vosahlik**

ASTM C39 Compressive Strength of Concrete Cylinders

Specimen Identification

CTLGroup Identification	Dowel Test A	Dowel Test B
Client Identification	N/A	N/A
Casting Date	4/23/2020	4/23/2020
Test Date / Time	5/7/2020	5/7/2020
Loading Rate, psi/sec	35	35

Concrete Description

Concrete Age at Test, days	14	14
Moisture Condition at Test	SSD	SSD
Curing Conditions (Temp/RH)	74°F/100% RH	74°F/100% RH
Cylinder End Preparation	Ground	Ground

Concrete Dimensions

Diameter 1, in.	4.01	4.03
Diameter 2, in.	4.01	4.02
Length, in.	7.85	7.86
Average Diameter, in.	4.01	4.03
Length / Diameter (L/D)	1.96	1.95
Cross-Sectional Area, in^2	12.63	12.76

Compressive Strength and Fracture Pattern

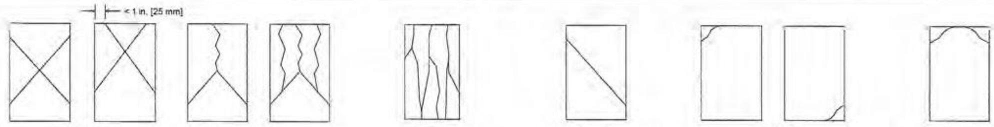
Maximum Load, lb	69,631	72,191
Compressive Strength, psi	5,510	5,660
Fracture Pattern	Type 1	Type 1

Average Compressive Strength of Companion Specimens	5,590 psi

Notes:

1. Samples fabricated by CTLGroup using the agreed upon mixutre supplied by a local ready mix operation.
2. Companion specimens were tested for the determination of compressive strength only.
3. The results specifically represent the tested samples.
4. This report may not be reproduced except in its entirety.

Schematic of Typical Fracture Patterns



Type 1 Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2 Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3 Columnar vertical cracking through both ends, no well-formed cones

Type 4 Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type l

Type 5 Side fractures at top or bottom (occur commonly with unbounded caps)

Type 6 Similar to Type 5 but end of cylinder is pointed

CTLGROUP

Client:	M. Snycer / ACPA	CTLGroup Project No:	052167
Project:	Evaluation of Dowel Bar Test Methods	CTLGroup Project Mgr.:	B. Birch
Contact:	M. Snyder	Technician:	W. Demharter
Date Reported:	May 21, 2020	Approved:	J. Vosahlik

ASTM C39 Compressive Strength of Concrete Cylinders
ASTM C469 Static Modulus of Elasticity of Cylindrical Concrete Specimens

Specimen Identification

CTLGroup Identification	Companion A	Companion B	Companion C
Client Identification	N/A	N/A	N/A
Casting Date	4/23/2020	4/23/2020	4/23/2020
Test Date / Time	5/21/2020	5/21/2020	5/21/2020
Loading Rate, psi/sec	35	35	35

Concrete Description

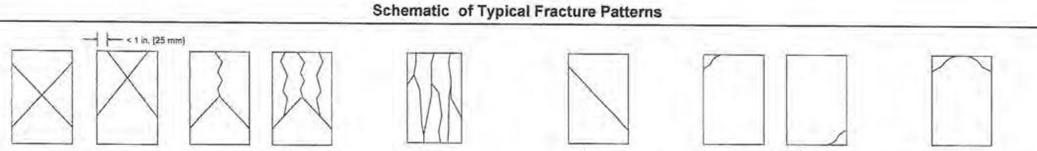
Concrete Age at Test, days	28	28	28
Moisture Condition at Test	SSD	SSD	SSD
Curing Conditions (Temp/RH)	74°F/100% RH	74°F/100% RH	74°F/100% RH
Cylinder End Preparation	Ground	Ground	Ground
Concrete Dimensions			
Diameter 1, in.	4.01	4.03	4.01
Diameter 2, in.	4.01	4.01	4.00
Length, in.	7.87	7.88	7.83
Average Diameter, in.	4.01	4.02	4.01
Length / Diameter (L/D)	1.96	1.96	1.95
Cross-Sectional Area, in^2	12.63	12.69	12.63
Compressive Strength and Fracture Pattern			
Maximum Load, lb	80,390	81,770	79,261
Compressive Strength, psi	6,370	6,440	6,280
Fracture Pattern	Type 1	Type 1	Type 1

Average Compressive Strength of Companion Specimens	6,360 psi

Notes:

1. Samples fabricated by CTLGroup using the agreed upon mixutre supplied by a local ready mix operation.
2. Companion specimens were tested for the determination of compressive strength only.
3. The results specifically represent the tested samples.
4. This report may not be reproduced except in its entirety.

Schematic of Typical Fracture Patterns



Type 1 Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2 Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3 Columnar vertical cracking through both ends, no well-formed cones

Type 4 Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type I

Type 5 Side fractures at top or bottom (occur commonly with unbounded caps)

Type 6 Similar to Type 5 but end of cylinder is pointed

CTLGROUP

Client:	M. Snycer / ACPA	CTLGroup Project No:	052167
Project:	Evaluation of Dowel Bar Test Methods	CTLGroup Project Mgr.:	B. Birch
Contact:	M. Snyder	Technician:	W. Demharter
Date Reported:	May 21, 2020	Approved:	J. Vosahlik

ASTM C39 Compressive Strength of Concrete Cylinders
ASTM C469 Static Modulus of Elasticity of Cylindrical Concrete Specimens

Specimen Identification

CTLGroup Identification	Modulus A	Modulus B	Modulus C
Client Identification	N/A	N/A	N/A
Casting Date	4/23/2020	4/23/2020	4/23/2020
Test Date / Time	5/21/2020	5/21/2020	5/21/2020
Loading Rate, psi/sec	35	35	35

Concrete Description

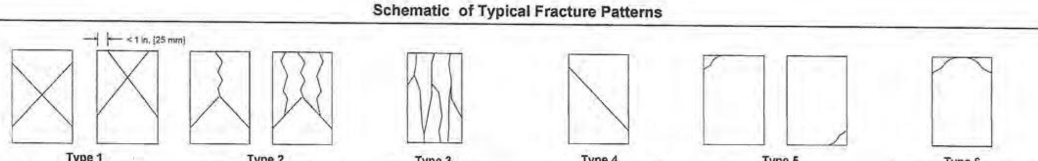
Concrete Age at Test, days	28	28	28
Moisture Condition at Test	SSD	SSD	SSD
Curing Conditions (Temp/RH)	74°F/100% RH	74°F/100% RH	74°F/100% RH
Cylinder End Preparation	Ground	Ground	Ground
Concrete Dimensions			
Diameter 1, in.	4.00	4.03	4.02
Diameter 2, in.	4.00	4.02	4.02
Length, in.	7.88	7.84	7.89
Average Diameter, in.	4.00	4.03	4.02
Length / Diameter (L/D)	1.97	1.95	1.96
Cross-Sectional Area, in^2	12.57	12.76	12.69
Compressive Strength and Fracture Pattern			
Maximum Load, lb	79,461	81,421	80,002
Compressive Strength, psi	6,320	6,380	6,300
Fracture Pattern	Type 1	Type 1	Type 1
Chord Modulus of Elasticity ksi	4,700	4,650	4,750

Average Compressive Strength of Modulus Specimens	6,340 psi
Average Compressive Strength of Companion & Modulus Specimens	6,350 psi
Average Elastic Modulus	4,700 ksi

Notes:

1. Samples fabricated by CTLGroup using the agreed upon mixutre supplied by a local ready mix operation.
2. Compressive strength of modulus samples A, B and C was determined after obtaining strain values for the modulus of elasticity.
3. The results specifically represent the tested samples.
4. This report may not be reproduced except in its entirety.

Schematic of Typical Fracture Patterns



Type 1 Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2 Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3 Columnar vertical cracking through both ends, no well-formed cones

Type 4 Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type I

Type 5 Side fractures at top or bottom (occur commonly with unbounded caps)

Type 6 Similar to Type 5 but end of cylinder is pointed

APPENDIX C. EXCERPT FROM CP TECH CENTER *GUIDE TO DOWEL LOAD TRANSFER SYSTEMS FOR JOINTED CONCRETE ROADWAY PAVEMENTS* (APPENDIX B – DESIGN FACTORS AFFECTING DOWEL BEARING STRESS AND FAULTING)

Appendix B - Design Factors Affecting Dowel-Concrete Bearing Stress (and Faulting)

To determine critical dowel-concrete bearing stress first requires identification of the portion of the design load that is carried by the critical (most heavily loaded) dowel.

The total shear load carried by a dowel group cannot be more than 50 percent of the applied load (which corresponds to 100 percent deflection load transfer conditions) and is a function of many factors, including the spacing, length, and diameter (or other section characteristics) of the dowels, thickness of the slab, width of the joint (which influences the behavior of the dowel system), stiffness of the supporting pavement layers, and "looseness" in the dowel bars (due to initial conditions and the effects of repeated loads). Studies by Tabatabaie (1978) and others have established that, for design purposes, values of 40 to 50 percent transferred load are appropriate. Heinrichs et al. (1987) found that this value is generally between 41 and 43 percent.

Friberg (1938) studied the theoretical behavior of dowels in rigid pavements and concluded that all dowels within a distance of 1.8ℓ of the point of load application (where ℓ is the radius of relative stiffness of the pavement-foundation system) would carry a portion of the load, with the magnitude of load carried being inversely proportional to the distance from the applied load. Westergaard (1925) had previously defined the radius of relative stiffness as follows:

$$\ell = (E_c h^3/12k(1 - \mu^2))^{0.25}$$

where E_C is the concrete modulus of elasticity, k is the modulus of foundation support (k-value), and μ is the concrete Poisson's ratio. For typical concrete slabs (thickness ranging from 8 to 12 in. and elastic modulus ranging from 3 to 6 million psi) constructed on granular subbases and subgrade soils with an effective k of 200 psi/in., the radius of relative stiffness ranges from about 28 to 45 in.

The introduction of finite element methods in the late 1970s offered a new tool for analyzing concrete pavement joints, and several researchers (Tabatabaie 1978, Tabatabaie et al. 1979, and Barenberg and Arntzen 1981) re-examined the distribution of loads at the pavement joint and found that the distribution of shear forces should be restricted to 1.0ℓ or less to reflect values computed using finite element analyses. This revised distribution assigns a much higher load to the critical dowel and results in higher bearing stresses. Heinrichs et al. (1987) confirmed these findings and further stipulated that the figure should decrease to about 0.6ℓ as the load approaches the slab corner. Figure B1 illustrates how the effect of the design load on the critical dowel can be estimated using the information above.

Once the load on the critical dowel has been determined, the bearing stress can be computed using an equation developed by Friberg (1940) based on work done by Timoshenko and Lessels (1925):

$$\sigma_b = Ky_0 = KP_t(2 + \beta z)/4\beta^3 E_d I_d$$

where K = modulus of dowel-concrete interaction (similar to k-value for soils), which is typically assumed to be 1,500,000 psi/in.; y_0 = deformation in the concrete under the dowel at the joint face; P_t = the magnitude of the transferred load in this dowel; z = joint width at the dowel bar; E = modulus of elasticity of the dowel; I_d = moment of inertia of the dowel (= $\pi d^4/64$ for round dowels, where d is the diameter of the dowel); and β = the relative stiffness of the dowel embedded in the concrete and is computed as follows:

$$\beta = (Kd/4E_d I_d)^{0.25}$$

Assumptions:

Wheel load = 9,000 lb

Transferred load = 42 percent of applied load (P_t = 9000x0.42 = 3,780 lb/wheel)

Dowel spacing, s = 12 inches

Slab thickness, h = 10 inches

Effective modulus of subgrade support = 200 psi/in.

PCC Modulus of elasticity = 4.0×10^6 psi

PCC Poisson's Ratio = 0.17

Radius of Relative Stiffness, $\ell = (E_c h^3/12k(1 - \mu^2))^{0.25}$ = 36.19 in.

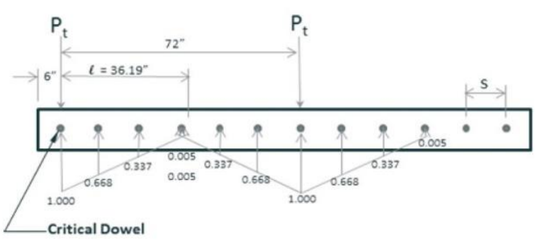


Figure B1. Sample computation of individual dowel shear loads within a dowel group

Calculation of effective dowels:

Dowel directly beneath load: 1.0 effective dowels

Dowels 12 in. from load: 24.19/36.19 = 0.668 effective dowels

Dowels 24 in. from load: 12.19/36.19 = 0.337 effective dowels

Dowels 36 in. from load: 0.19/36.19 = 0.005 effective dowels

Edge load is carried by 1.0 + 0.668 + 0.337 + 0.005 = 2.010 effective dowels

Mid-panel load is carried by 1.0 + 2(0.668) + 2(0.337) + 2(0.005) = 3.020 effective dowels

Critical dowel carries 3780(1.000/2.010) = 1881 lb

Adjacent dowel carries 3780(0.668/2.010) = 1256 lb

Other dowel loads can be computed similarly.

From these equations, it is clear that dowel bearing stress is directly proportional to the magnitude of the transferred load, as well as the joint width and the modulus of dowel-concrete interaction. It can also be inferred that bearing stress increases with decreasing dowel elastic modulus and moment of inertia (or diameter, for round dowels). Because bearing stress is directly related to y_0 (deformation in the concrete under the dowel at the joint face), factors that increase bearing stress also increase differential deflection across the joint and increase the potential for pumping and faulting. Furthermore, repeated applications of higher-bearing stresses result in more rapid increases in dowel looseness, which further increase differential deflections and potential for pumping and faulting.

While ACI Committee 325 (Concrete Pavements) currently makes no recommendations concerning limits for dowel bearing stress, in 1956 they published a document containing the following recommendation (which resulted in factors of safety of 2.5 to 3.2 against bearing stress-related cracking) (American Concrete Institute 1956):

$$f_b = f'_c(4 - d)/3$$

where f_b = allowable bearing stress, f'_c = concrete compressive strength and d = dowel diameter (in.).

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