INTRODUCTION

In AC 150/5320-6F, pavement joints are categorized into three types: isolation (Types A, A-1), contraction (Types B, C, D), and construction (Types E, F) joints. Depending upon their design, the function of such joints is to control the stresses caused by expansion, contraction, and warping of the concrete.

Doweled joints, whether construction or contraction joints, depend primarily on the shear strength of the dowel and the bearing stress of the concrete to transfer the load. Their design is usually limited by the bearing strength of the concrete, which governs how loose the dowel becomes after repeated heavy loads. Doweled construction joints are currently the only type of construction joint allowed for airfield pavements with large aircraft operations. In general, the use of dowels plays an important role in ensuring good load transfer especially when the slab contracts at low temperatures and results in a tight joint.

Undoweled contraction (dummy) joints depend on aggregate interlock for load transfer. Dummy joints are very sensitive to the crack width opening and tend to perform better with short joint spacings.

Keyed joints use the geometry of the key shape to transfer the load across a joint by producing bearing and shear stresses in the male and female portions of the key. AC 150/5320-6E eliminated all keyed joints from the schedule of standard joint types due to a history of poor performance. However, European contractors have experienced some success with a sine-wave shaped keyway having three or four smooth shaped waves with approximately 1-2 inches amplitude. This sinusoidal detail is intended to encourage better construction joint face interlock, compared to the rectangular key cross section previously common in the U.S. By eliminating hard corners, the sinusoidal keyway shape also reduces stress risers than cause breakage.

In this plan, the following conventions will be used to designate joint types:

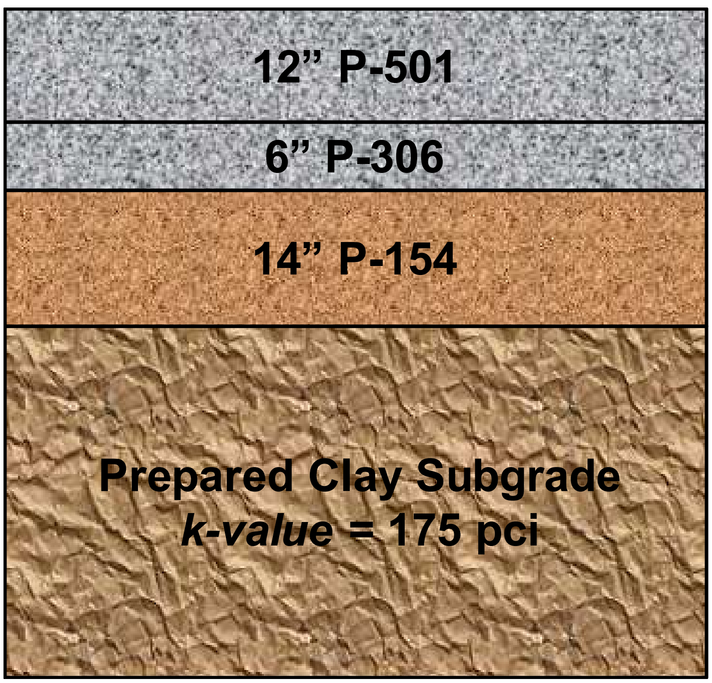
Type C: Doweled Contraction Joint
Type D: Undoweled (Dummy) Contraction Joint
Type E: Doweled Construction joint
Type K: Keyed (Sinusoidal) Construction Joint

1. TEST PAVEMENT AND INSTRUMENTATION

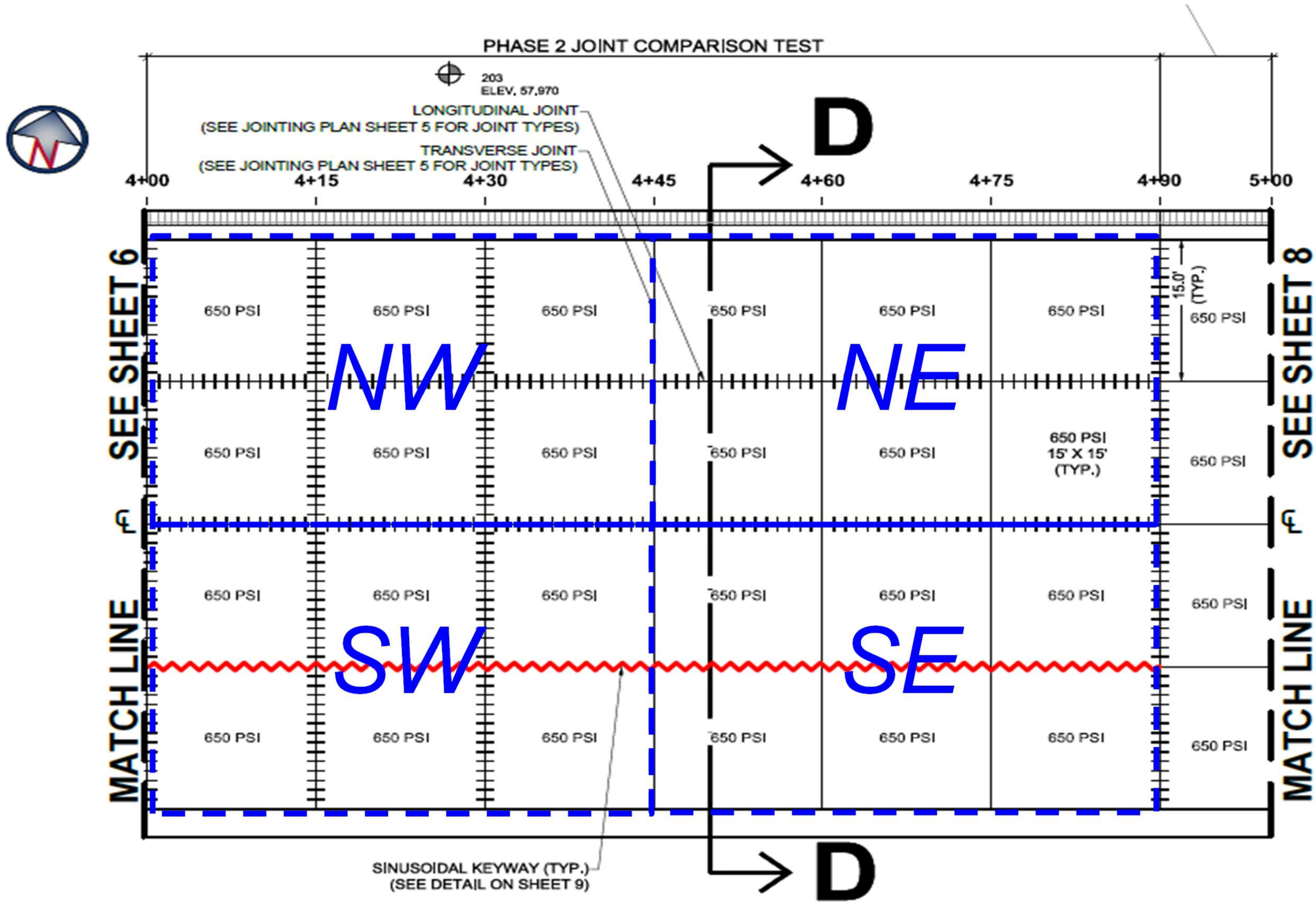
The CC8 Joint Comparison Test (JCT) area covers an area 90 feet long by 60 feet wide between stations 4+00 and 4+90. Figure 1(a) shows the pavement cross section for both north and south. The section consists of: 12 inches P-501 concrete; on 6 inches P-306 lean concrete base; on 14 inches P-154 granular subbase; supported on a prepared clay subgrade with an average California Bearing Ratio (CBR) value of 7.6. The design flexural strength (R) of the P-501 concrete mix was 650 psi. Based on plate load tests at the top of the subgrade, the average modulus of subgrade reaction (k-value) was 154 pci for the north, and 197 pci for the south. Figure 1(a) shows the profile view. There is a total of (24) 15-ft by 15-ft slabs, (12) on the north and (12) on the south. The (24) slabs are divided into (4) groups as indicated by the blue dashed lines in Figure 1(b).

Each group represents a different combination of longitudinal and transverse joint types. All dowels are 0.75 inches in diameter.

Figure 2 (a, b) show the locations of all subgrade, subbase and concrete slab sensors. Vertical movement of slab corners relative to the subbase is monitored by eddy current sensors (ECS). The ECSs are intended to operate both in static mode (to monitor long-term upward movement of slab corners) and in dynamic mode (to record transient responses to vehicle loads). Pairs of embedded strain gages (EG) were installed along longitudinal and transverse edges of 16 slabs to measure strain responses near the top (odd numbered gages) and bottom (even numbered gages) of the instrumented slabs. Rebar chairs ensured that strain gages were set at the proper height (gage center 1 in. above the slab bottom and 1 in. below the slab top). Thermocouple trees were installed in two slabs to monitor slab temperature gradients. Each tree consists of three thermocouples to measure temperature at the bottom, middle and top of the slab. In addition, moisture sensors were installed at two locations to monitor the subgrade soil moisture content. The moisture sensors were located at 6 in below the subgrade surface during the subgrade preparation.



a) Profile view



(b) Plan view

Figure 1. (a, b) As-built Test Pavement Structure.

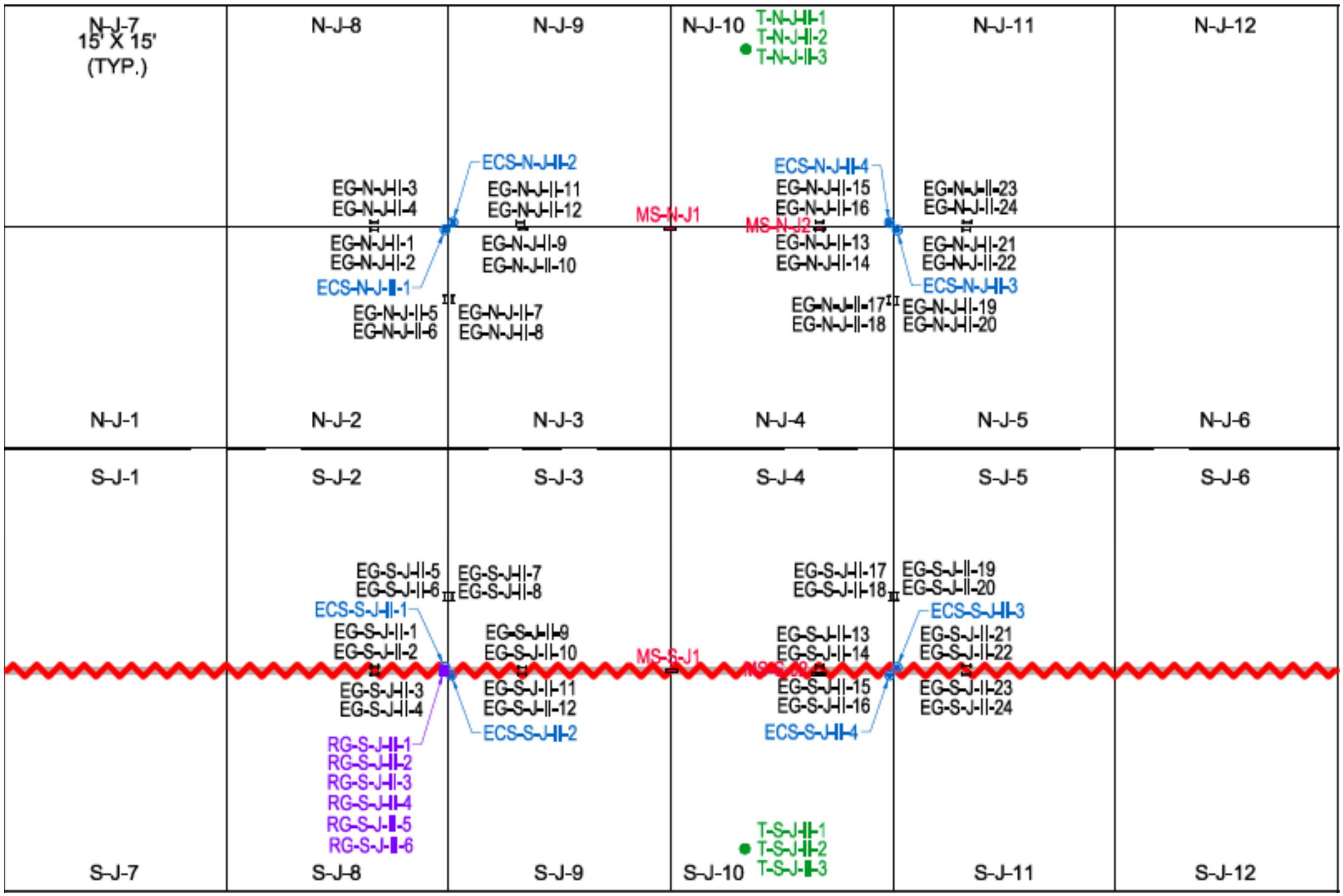


Figure 2a. Instrumentation Plan View.

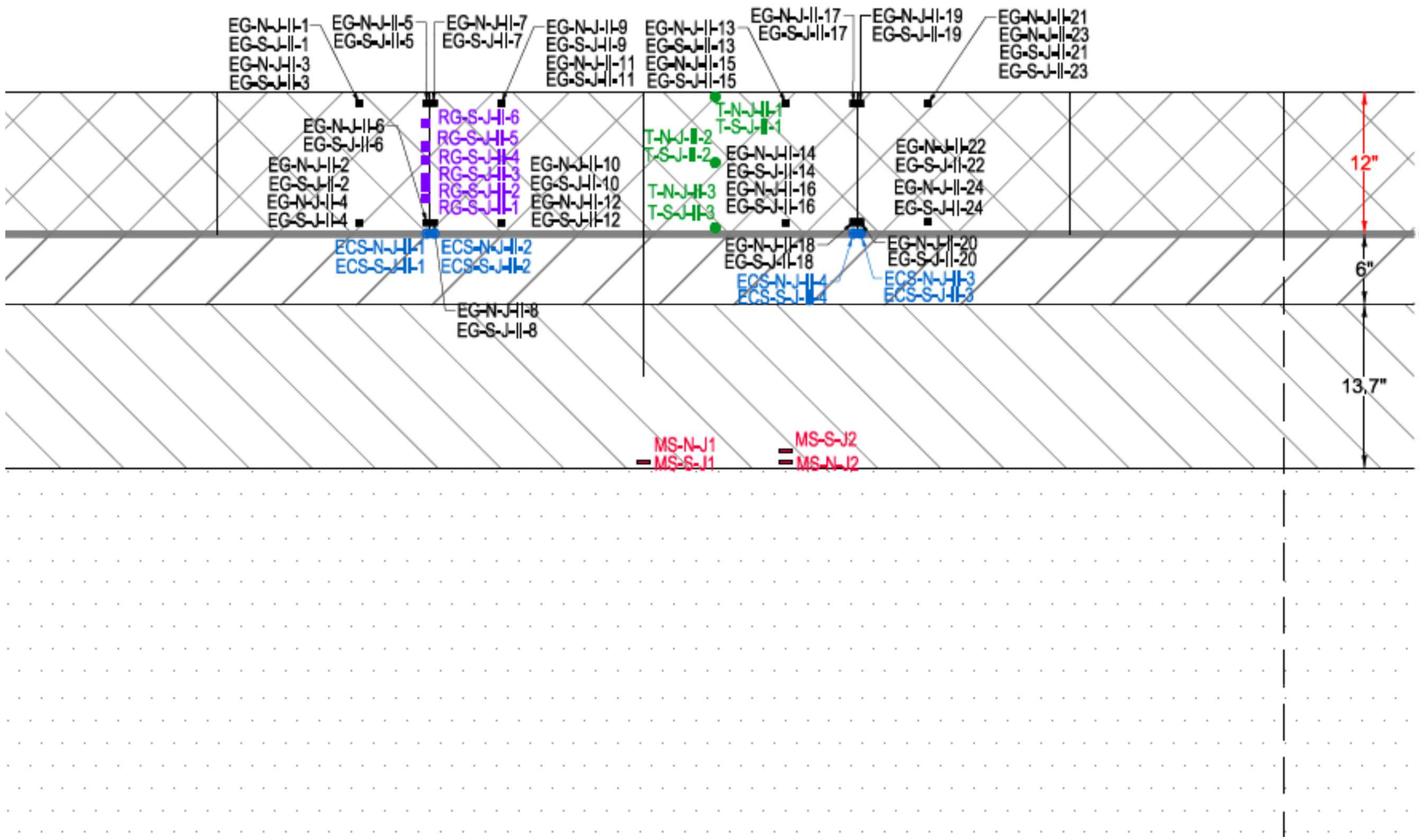


Figure 2b. Instrumentation Profile View.

2. PERFORMANCE EVALUATION

2.1 STRUCTURAL DETERIORATION

Throughout the traffic test, the structural performance of the test pavements will be monitored and quantified by means of the Structural Condition Index (SCI). SCI is a modification of the Pavement Condition Index (PCI) for Airports (rigid) method following ASTM D 5340 (ASTM 2012). Like PCI, SCI is based on visual inspection of the pavement surface and identification of standard distresses. The difference is that in the SCI only distresses related to structural loading are counted, while environmental and construction/material-related distresses are disregarded.

2.2 LOAD TRANSFER

The concept of load transfer at rigid pavement joints is fundamental to airfield rigid pavement design. Figure 3 shows a conceptual view of the mechanism of load transfer at a joint. In airfield applications, the following two definitions have been used most commonly:

$$Load\ Transfer\ Efficiency\ for\ Deflection\ (LTE_{\delta}) = 100\left({}^{\delta_U}/_{\delta_L}\right) \tag{1}$$

$$Load\ Transfer\ Efficiency\ for\ Stress\ (LTE_{\sigma}) = 100\left({}^{\sigma_U}/_{\sigma_L}\right) \tag{2}$$

Where:

δ_L = Deflection of the loaded side of the joint
δ_U = Deflection of the unloaded side of the joint
σ_L = Bending stress at the joint in the loaded slab
σ_U= Bending stress at the joint in the unloaded slab

If load transfer is assumed to be by shear alone (i.e., load transfer by moment is negligible), then the sum of the stress on the loaded slab (σ_L) and the stress on the unloaded slab (σ_U) is equal to the maximum edge stress. (This assumption also disregards any effect of slab curling.) The FAA design procedure assumes 25 percent of the load applied to an edge is transferred at the joint to an adjacent unloaded slab. This assumption effectively reduces the edge stress in the loaded slab by 25 percent compared to a free edge condition, allowing for a reduced slab thickness. Current practice, for pavements with heavy traffic applications, is to use dowels to ensure that a sufficient level of load transfer is maintained throughout the pavement's life. Stress in PCC slabs cannot be measured directly, only estimated from strain measurements. If the modulus of loaded and unloaded slabs is assumed to be the same, then strain gage readings can be used to get estimates of stress, or change in stress, which is directly related to bending strain.

The LTE_{δ} is different than LTE_{σ}, and has evolved more with a focus on measuring joint deflections, which can be easily accomplished with equipment such as the Falling Weight Deflectometer (FWD) and Heavy Weight Deflectometer (HWD). Many airport authorities collect deflection data on their pavement systems for pavement management, rehabilitation evaluation, and forensic evaluation purposes, and they consider deflection data as important as pavement condition and distress data. The FWD/HWD device is a stationary dynamic load pulse type load test. This dynamic pulse can be considered to be somewhat like a rolling wheel load. When considering the typical load pulse duration for the FWD/HWD, it is similar to the load pulse duration that would be generated by an aircraft wheel moving at about 40 mph.

In this study, the load transfer (LTE_δ and LTE_σ) at both longitudinal and transverse joints will be monitored by two methods:

1. calculate LTE_δ from HWD deflections; and
2. calculate LTE_σ from dynamic strain responses in EGs under moving gear loads.

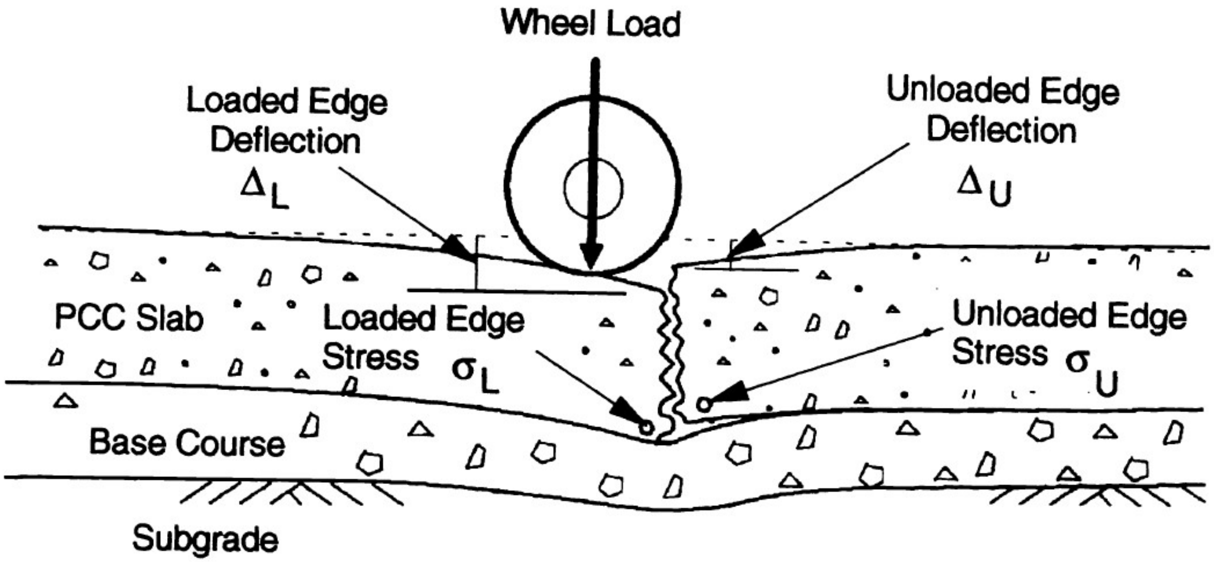


Figure 3. Concept of Load Transfer at Joint.

2.2.1 Procedure for Measuring Load Transfer Efficiency (Deflection) by HWD

Perform all HWD tests using the FAA KUAB Model 150 tester with a four-drop loading sequence beginning with a 36,000-lb seating load. The drop loads will be: 12,000 lbs., 24,000 lbs., and 36,000 lbs. Place the loading plate at a transverse joint with sensors D_0 and D_1 on opposite sides of and equidistant from the transverse joint (Figure 4a). Then reposition the HWD, again such that sensors D_0 and D_2 are equidistant from the joint (Figure 4b). Repeat the same positioning sequence at the longitudinal joint. Locations for HWD testing are marked by the blue dots in Figure 5.

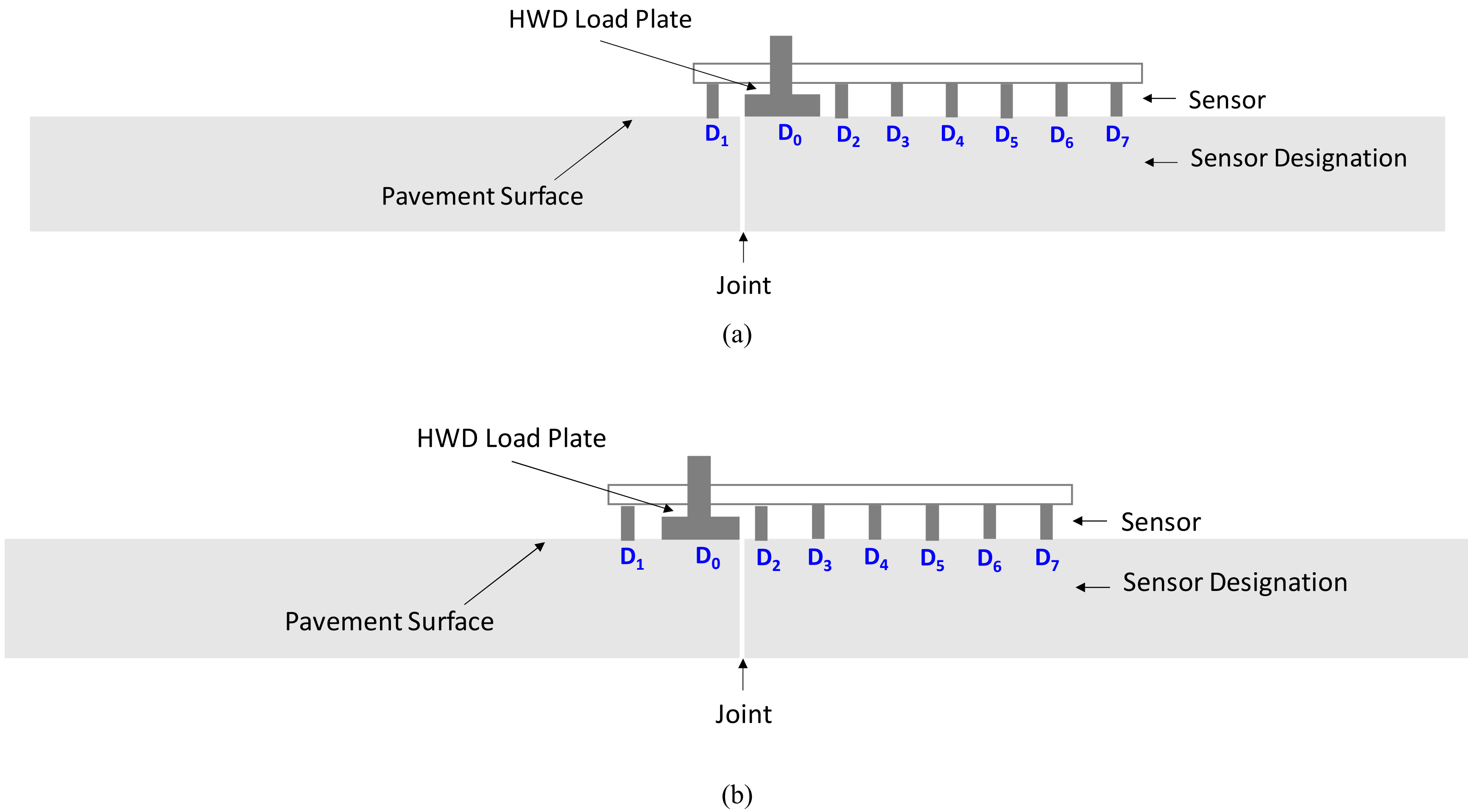


Figure 4. HWD Sensor Location

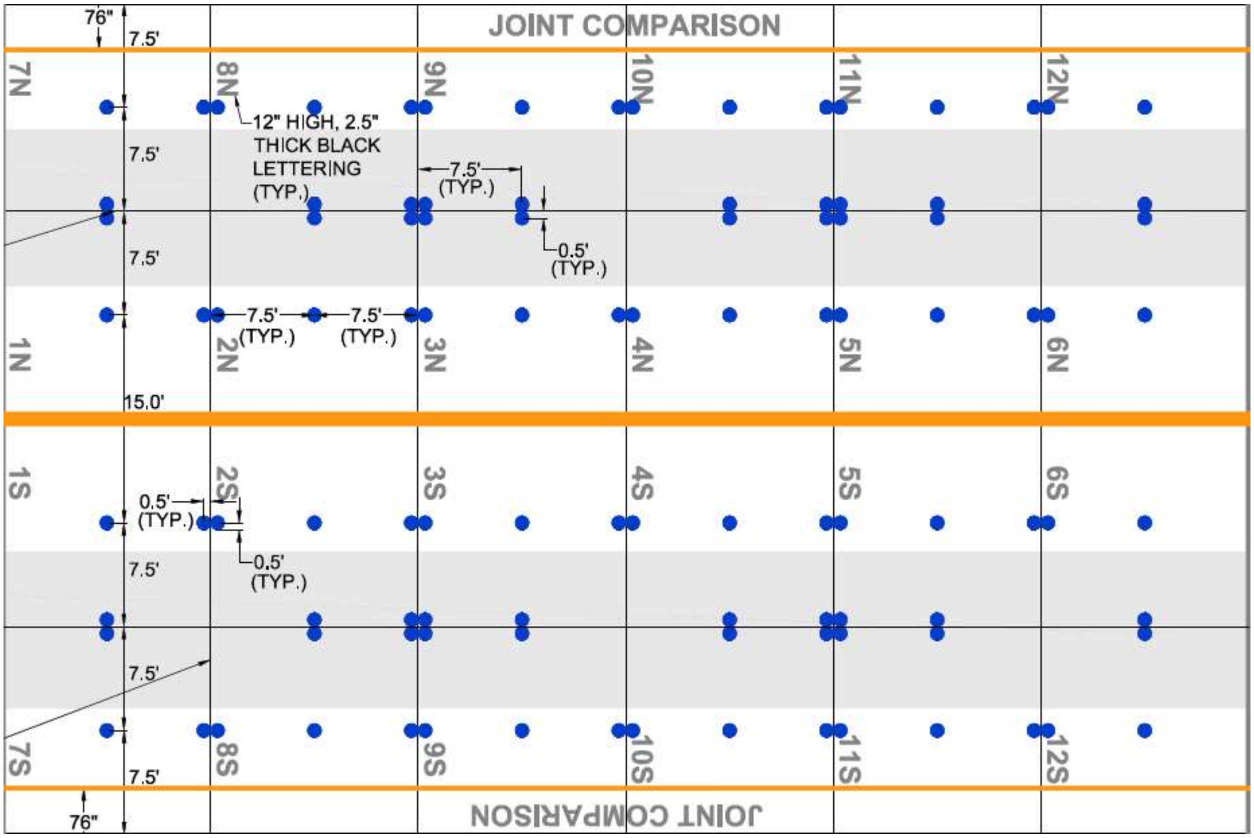


Figure 5. HWD Test Locations.

Compute load transfer efficiency for deflection (LTE_δ) from HWD deflections using Equation 1. Calculate the estimated joint stiffness (k_J) using the geometric method, as described in IPRF Report 05-02, *Joint Load Transfer in Concrete Airfield Pavement*:

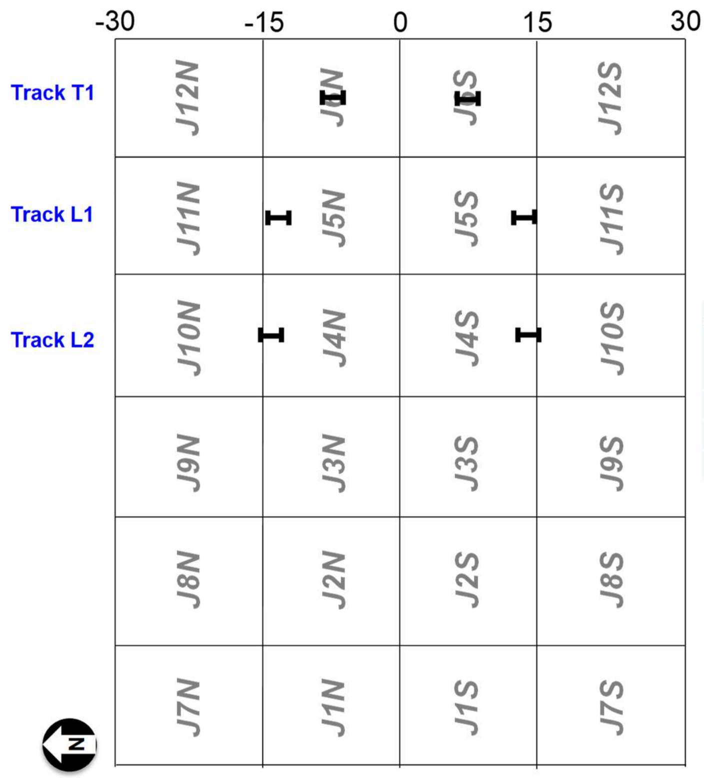
$$k_J = P(LTE_\delta)/[(1 + LTE_\delta)(D_{-6} - D_6)(1 + i_{\%})\left(66 + \frac{60D_{66}}{D_6 - D_{66}}\right)] \quad (3)$$

Where:

P = HWD drop load
D_i = Deflection at sensor distance from the joint
$i_{\%}$ = Percentage increase factor needed to project the sensor readings out to the joint.

2.2.2 Procedure for Measuring Load Transfer Efficiency (Stress) Under Moving Gear Loads

It is preferable to consider strain responses from bottom EGs (even ID) in the calculation of stress-based LTE, but the top EGs at the same location will be used as replacement if their paired ones fail during trafficking. Use the NAPTV to induce strains in EGs in a rolling wheel test. The vehicle speed is 2.5 mph and NAPTV travels in both W→E and E→W directions. With a wheel load of 36,000 lbs., position the NAPTV such that the outer wheel of the carriage modules tracks directly above the EGs of interest. For transverse EGs use track T1. For longitudinal EGs on the inner slabs use track L1. For longitudinal EGs on the outer slabs, use track L2. In general, these lateral gear positions are not on a standard wander track, as illustrated in Figure 6. However, track L1 does correspond to a standard wander track, i.e., track 0. Take the transverse joint at STA 430 for example. For track T1 on the north side, first take the maximum tensile strain from EG-N-J-II-6 in the loaded slab (N-J-2) when the outer wheel is directly above the gage. At the same time, take the maximum tensile strain from EG-N-J-II-8 in the unloaded slab (N-J-3). Then calculate the bending stress in loaded and unloaded slab, assuming the elastic modulus of concrete from baseline PSPA measurement. Lastly, calculate the LTE_{σ} using Equation 2.



Track #	Joint Evaluation	Carriage Centerline Location, ft	
		North	South
T1	Transverse	-7.500	7.500
L1	Longitudinal	-12.250	12.250
L2	Longitudinal	-13.250	13.250

Figure 6. Gear Tracks for Joint Evaluation.

3. TRAFFIC TEST

3.1 ESTIMATE INITIAL WHEEL LOAD

FAARFIELD 1.42 analysis was used to obtain strains and estimated failure passes. The same pavement structure was used for both north and south. The following conditions were assumed:

- as-built pavement structure (Fig. 1(a));
- 3 gear configurations (D, 2D, 3D);
- range of wheel loads (47,500 lbs. – 67,500 lbs.)
- subgrade k = 175 pci (average of north and south, Fig. 1(a)).
- R = 710 psi (average 270-day field-cured beam strength)

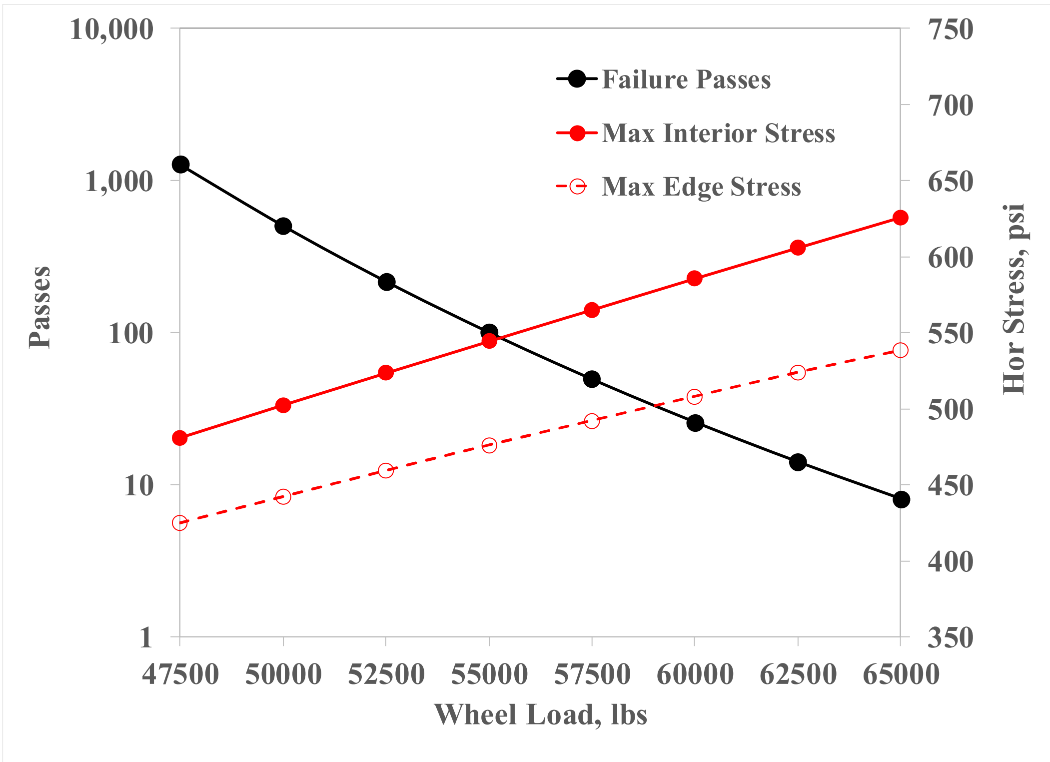
See a summary of FAARFIELD predictions at different wheel loads in Table 1. Computed stress ratios (σ/R) range between 0.7 and 0.9. FAARFIELD computes two values of maximum slab stress: the edge stress, assuming the gear is positioned at the joint; and the interior stress, assuming the gear is positioned at the slab center. The number of failure passes from FAARFIELD is based on the larger of 0.75 times the edge stress, or 0.95 times the center stress. From table 1, observe that:

- The critical value of horizontal stress (edge vs. interior) depends on the gear configuration.
- The D gear has a higher risk of joint deterioration due to larger edge stresses.

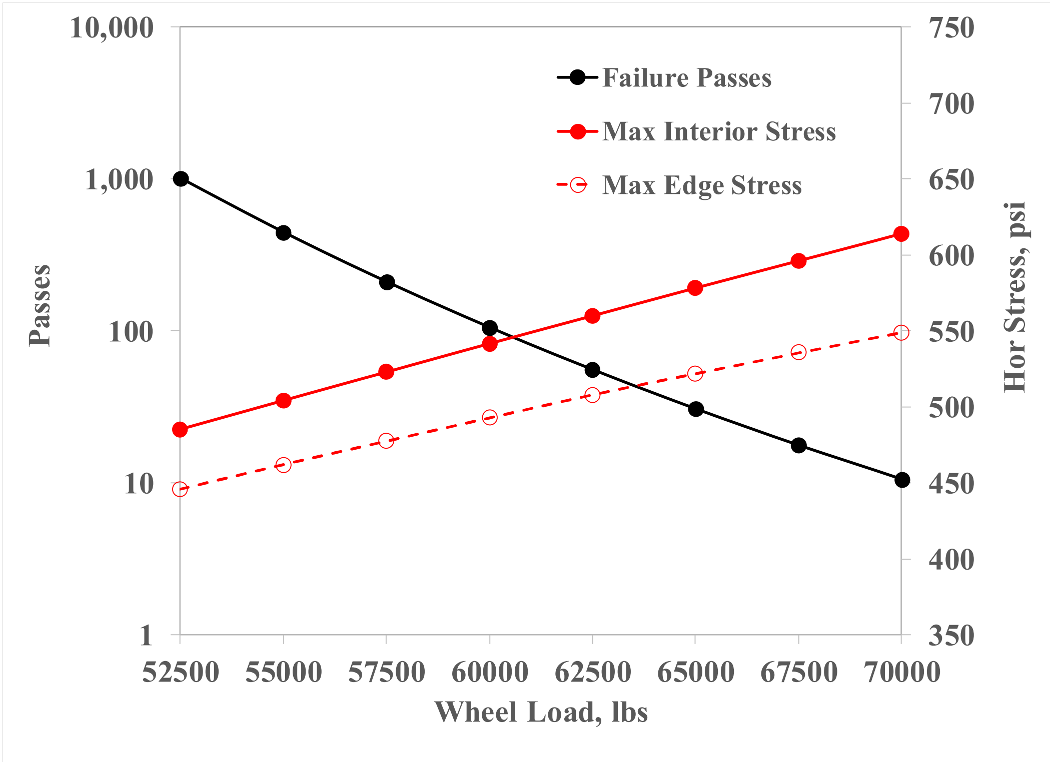
Table 1. FAARFIELD Predictions.

Gear	Wheel Load, lbs	Max Hor Stress, psi		Failure Passes
		Edge	Interior	
3D	47500	425	481	1285
	50000	442	503	508
	52500	459	524	218
	55000	476	545	101
	57500	492	565	50
	60000	508	586	26
	62500	524	606	14
	65000	538	626	8
2D	50000	429	466	2490
	52500	446	485	1016
	55000	462	504	448
	57500	478	523	212
	60000	493	542	106
	62500	508	560	56
	65000	522	578	31
	67500	536	596	18
	70000	549	614	11
D	50000	494	453	734
	52500	513	472	329
	55000	532	490	157
	57500	551	508	79
	60000	570	526	41
	62500	587	544	24
	65000	604	562	15
	67500	620	579	9

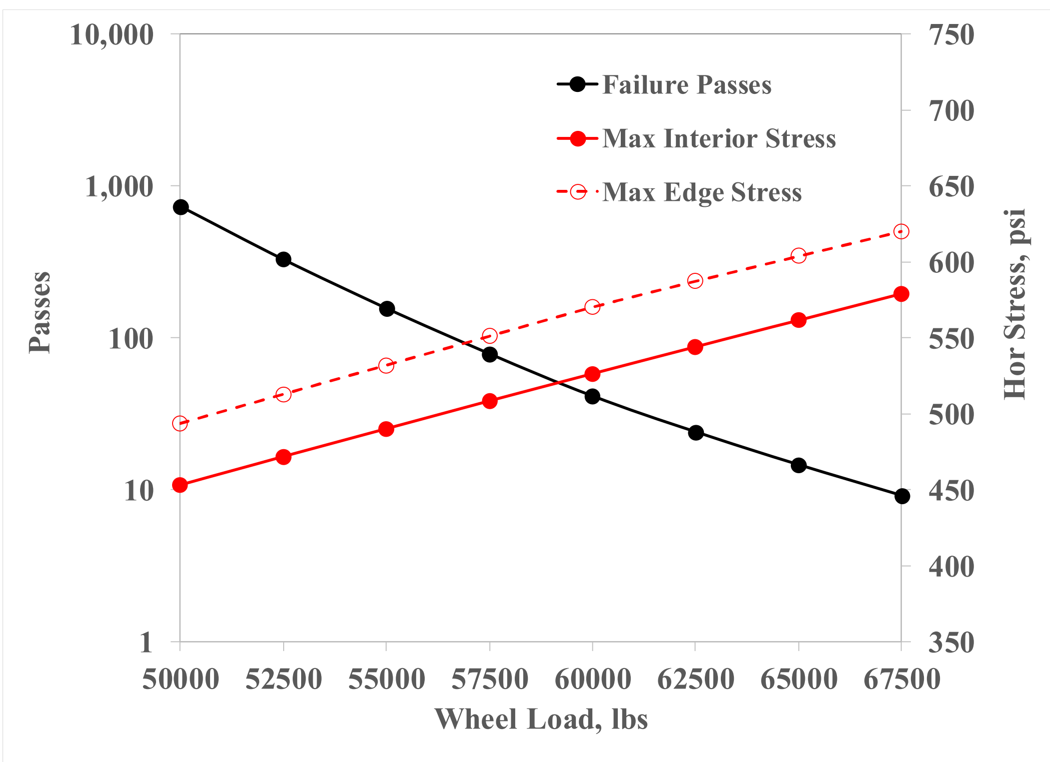
Assume target failure passes = 1000, with the understanding that the actual pavement life may be greater. Failure in FAARFIELD is defined as SCI = 80. However, additional traffic is anticipated to bring slabs to a full-failure or shattered-slab condition. Any difference in as-built versus asdesigned structure may cause significant over- or under-prediction of life. In addition, the FAARFIELD design model contains a number of conservative assumptions (fully unbonded slab-base interface, infinite subgrade depth) that may not be reflected in the as-built structure. Figure 7(c) suggests 50,000 lbs. as the initial wheel load for both north and south for 1000 passes to failure. This initial wheel load corresponds to stress ratio $\sigma/R = 0.7$. Therefore, use 50,000 lbs. as the initial wheel load for traffic tests.



(a) 3D



(b) 2D



(c) D

Figure 7. Life and Stress Predictions from FAARFIELD.

3.2 GEAR CONFIGURATION

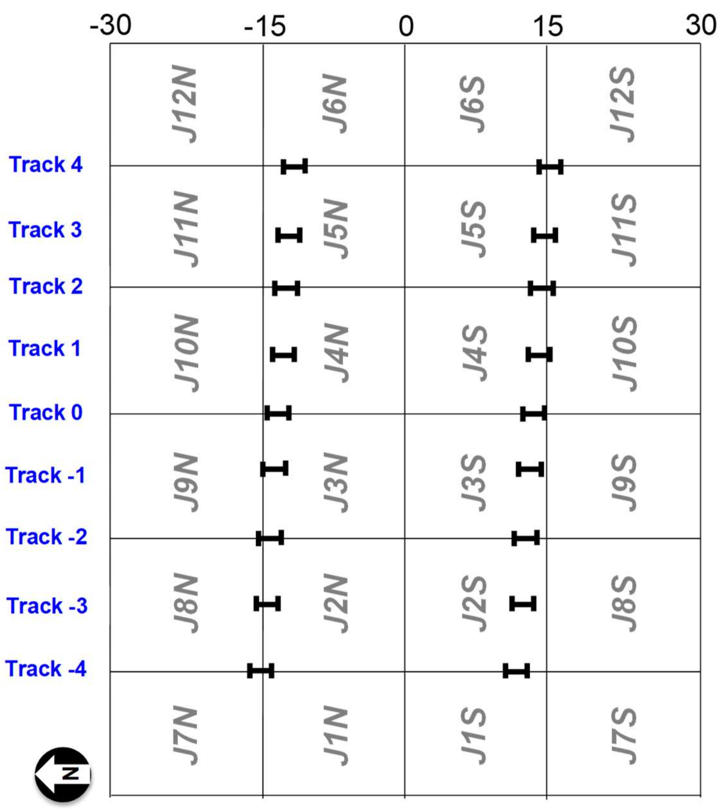
Figure 7 shows that the edge stress is maximum for the D configuration, but not necessarily for the 2D or 3D configurations. For a joint comparison test, it is desirable to have the maximum stress always at the joint. Therefore, use the D configuration.

- Traffic both north and south using the dual (D) gear.
- Inflate tires to 220 psi for all traffic tests.

3.3 WANDER PATTERN

The lateral gear position affects both the magnitude and the location of critical responses in rigid pavements. Maximum edge stresses occur when the loads are placed on or very close to a joint, and the stresses diminish rapidly as the load is moved away from the joint.

Conduct all traffic tests using the standard NAPTF wander pattern in Figure 8. Track 0 is the center of the traffic distribution. Track 0 places the outside edge of the outer wheel on the doweled longitudinal joint (north and south). Tracks are spaced 10 inches apart.



Track No.	Carriage Centerline Location, ft	
	North	South
-4	-15.662	8.838
-3	-14.809	9.691
-2	-13.956	10.544
-1	-13.103	11.397
0	-12.250	12.250
1	-11.397	13.103
2	-10.544	13.956
3	-9.691	14.809
4	-8.838	15.662

Figure 8. Wander Pattern for Trafficking.

4. TEST PROCEDURE

a. <u>General</u>. All traffic will be at 2.5 mph vehicle speed with nominal tire pressure 220 psi.

b. Wander Pattern. The wander pattern consists of 66 passes (Table A1), with each passage of the NAPTV to the east being counted as a pass, and the return to the west counting as a second pass. These 66 passes are arranged in 9 tracks, as shown in Figure 8. For Track 0, the outside tire of each dual aligns with the longitudinal joint centered within each test item. Detailed carriage positions for each pass for 1 full wander can be found in Table A1 in the appendix.
c. Slab Identification. All slabs shall be labelled as demonstrated in Figure 5.
d. HWD Location. Mark HWD test locations at the center of all 15'x15' slabs, slab edges and slab corners. See Figure 5.
e. Flexural Strength. Immediately prior to the seating load, conduct flexural strength tests on the field-cured beams cast during concrete placement. Follow ASTM C78. FAARFIELD shall then be re-run with new field *R* values to obtain more realistic calculation of failure passes.
f. Seating Loads. Traffic the test pavement (Table A2) using a two-wheel (dual) gear at a load of 10,000 pounds per wheel. Use the seating load pattern as shown in Figure 9. The seating load pattern consists of 21 tracks spaced every 10 inches to cover the pavement width (except for areas near the north and south shoulders that are out of range of the gear). Monitor slab vertical movements during seating using ECS deflection sensors, and note any effects of seating loads in the Daily Notes.
g. Baseline HWD and PSPA. After seating, perform HWD tests at all locations specified in step (d). Conduct HWD tests using a four-drop loading sequence beginning with a seating load, as in 3.2.1. Collect PSPA measurements from slab centers and ECS installed corners. Use the baseline HWD and PSPA measurements to backcalculate layer moduli. Subsequent tests will be referenced to the baseline to monitor slab curling and changes in support conditions.
h. Ramp-up Response Test. Conduct the ramp-up response test with the full wander pattern (Fig. 8, table A1). The purpose of this test is to make sure all systems are operating properly, and to assist in making the final decision about the wheel load to be used for the traffic test. Use D gear for both north and south side.
    1) Traffic 1 wander (66 passes) for both test items at the initial wheel load (50,000 lbs., or as determined in step (e)). Check to verify test items are not damaged. Record baseline sensor readings for dynamic sensors.
    2) Check that the maximum strain response for all sensors occurs on the expected track. Identify the maximum strain on the critical track for all EGs.
    3) Extrapolate from the maximum strain responses at slab top and bottom in (2) to the extreme fiber. Compare the extrapolated strains to the FAARFIELD calculations.
    4) Increase wheel load in increments of 2,500 lbs. Traffic only the critical tracks for both directions (W→E and E→W). Repeat step (3) until either peak slab top or bottom tensile strain = 90% of FAARFIELD calculations. An example of such practice is given in Figure 10, which was derived from the response test on CC8 Phase I Overlay Test Area. While both top (EG-S-O-II-9) and bottom (EG-S-O-II-4) extreme fiber strains linearly grew as the increase of wheel load, only the bottom strain reached 90% of FAARFIELD computed values. Therefore, the final wheel load for trafficking was determined as 55,000 lb.

i. Traffic Test.

1) Traffic both north and south test items using the D gear. Use the wheel load as determined in step (h). Continue trafficking until a single digit SCI condition is achieved on both sides. If a single digit SCI is attained on either test item, stop trafficking on that item, but continue trafficking on the other test item until the SCI is less than 10.
2) Joint Evaluation.
    - After every 10 wanders (about 1-day trafficking), following *3.2.2*, traffic both north and south side using D gear at the same wheel loading as 1) for 6 passes (W→E and E→W).
    - At the end of each trafficking week (about 40 wanders), conduct HWD measurements at the longitudinal and transverse joints as shown in Figure 4.

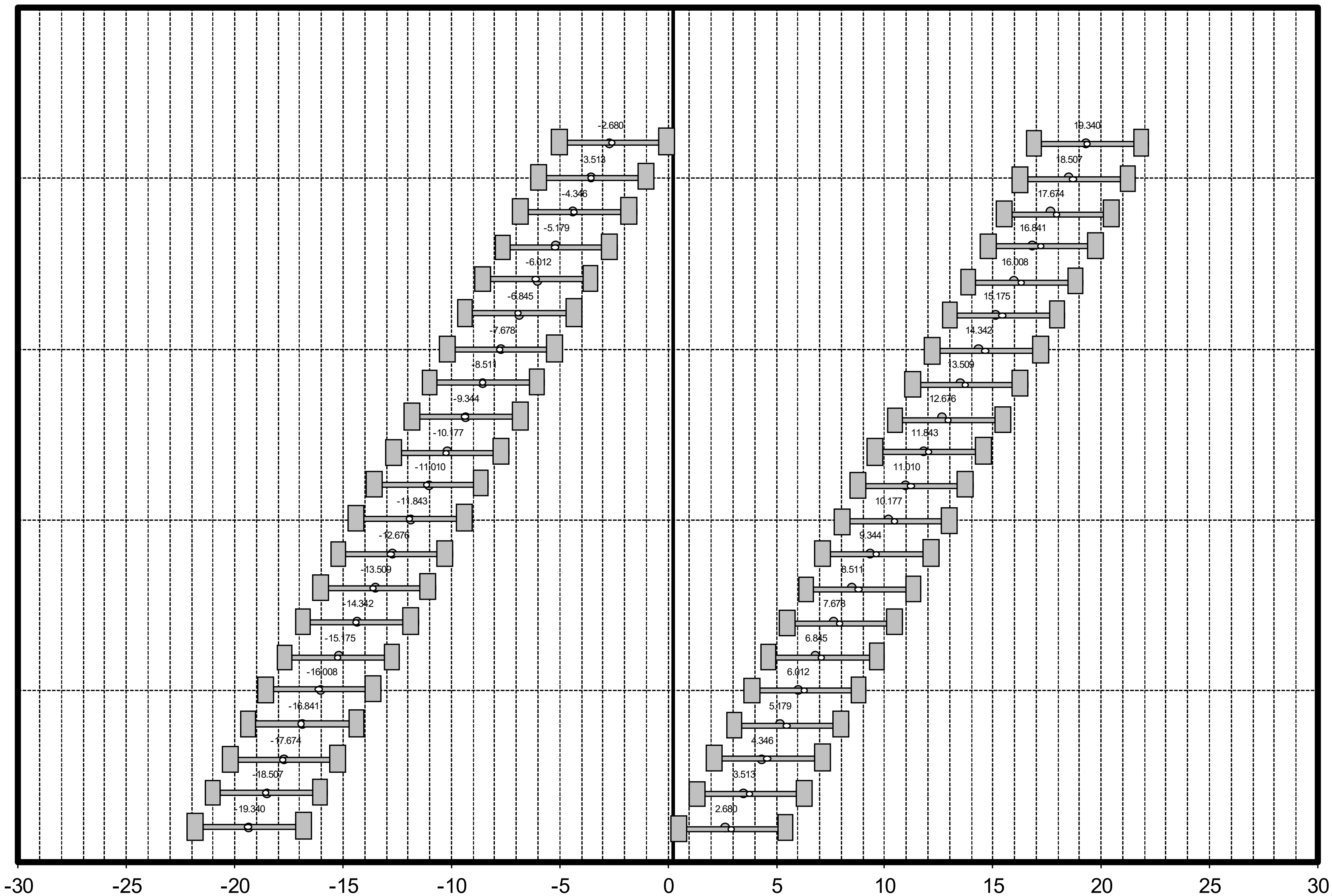
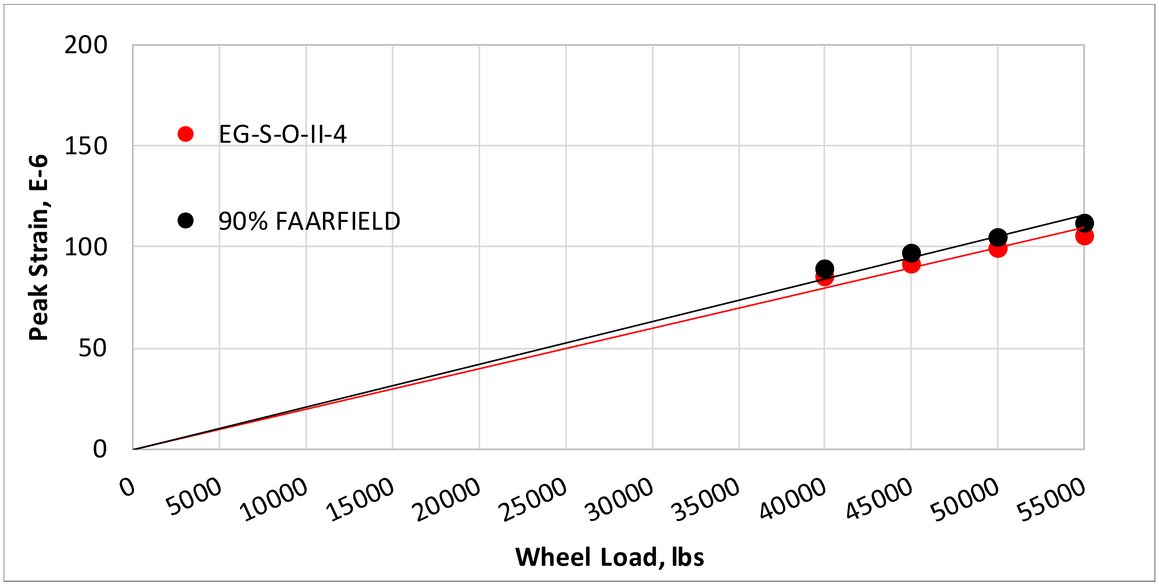
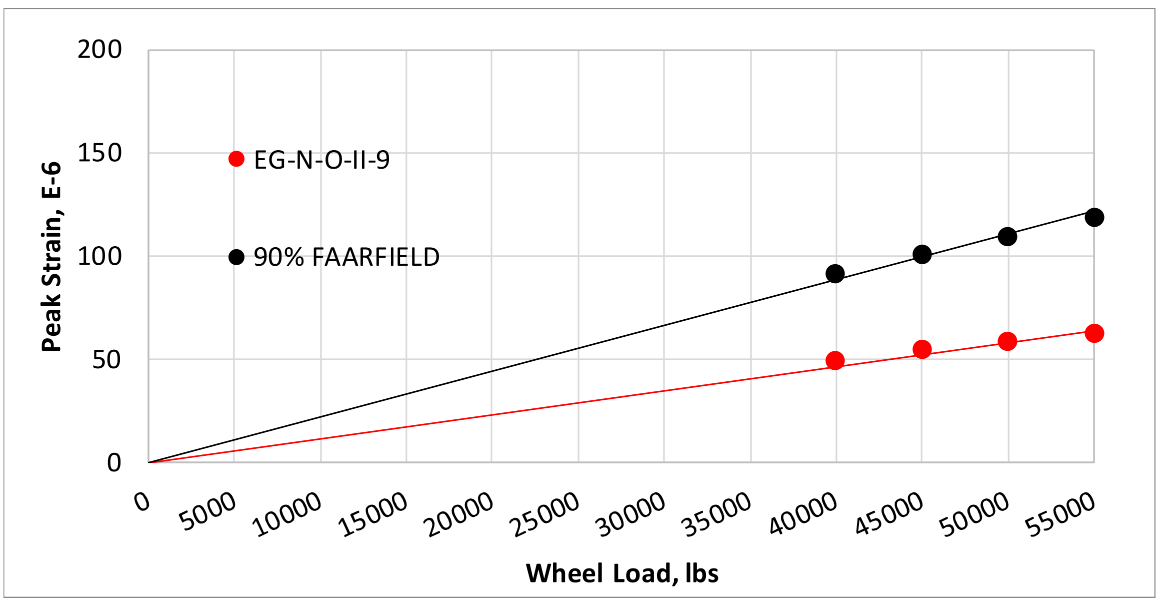


Figure 9. Seating Load Wander.



(a) Bottom EG



(b) Top EG

Figure 10. Ramp-up response test, 5,000 lbs wheel load increment, CC8 Phase II Overlay Test Area.

5. MONITORING

a. Dynamic Responses. Embedded strain gage (EG) and Eddy Current sensor (ECS) data will be collected through the SPUs. During traffic testing, the ViewData program will be utilized directly to monitor responses indicating rupture at gage locations. For subsequent data analysis, raw data files will be processed and stored.
b. Static Responses. Temperature and moisture data will be collected hourly.
c. Pavement Condition.

1) Manual Distress Survey. Conduct distress surveys on a daily basis for all 15×15' slabs. In addition, observe the test pavement informally after each wander and when the appearance of any new distress is noted. In accordance with ASTM D5340, longitudinal, transverse and diagonal cracking; corner breaks; intersecting cracks and shattered slabs; and shrinkage cracking will be considered. As needed, the surveys will be augmented with wire brushes, chalk markings, flashlights and other tools to ascertain the presence and pattern of very fine cracks. Cumulative plots of crack mapping will be prepared and submitted to the PI's on a daily basis. On these plots, the distresses will be color-coded to separate dates/passes of distress survey on which new distresses are observed.
2) SCI Calculation. After each distress survey, update pavement inspections in the PAVEAIR database and calculate a structural condition index (SCI).
3) In addition to 5.i.2, HWD and PSPA testing should be conducted on a weekly basis to detect any changes in pavement deterioration and support condition over time. These measurements shall be taken at the blue dots (see Figure 5). Both ECS data and the edge-to-center deflection ratios will provide information of any lift-off of the PCC slabs.

6. DATA STORAGE

a. Static Data: \\NAPTF\naptf\Static
b. Dynamic Data and Daily Notes: \\NAPTF\naptf\Trafficking

APPENDIX A—SUMMARY OF WANDER PATTERN

Table A1. Carriage positions for each pass for 1 full wander.

Pass Sequence No.	Direction	Track No.	Carriage Centerline Location, ft.	
			North	South
1	W→E	-4	-15.662	8.838
2	E→W	-4	-15.662	8.838
3	W→E	-2	-13.956	10.544
4	E→W	-2	-13.956	10.544
5	W→E	0	-12.250	12.250
6	E→W	0	-12.250	12.250
7	W→E	2	-10.544	13.956
8	E→W	2	-10.544	13.956
9	W→E	4	-8.838	15.662
10	E→W	4	-8.838	15.662
11	W→E	3	-9.691	14.809
12	E→W	3	-9.691	14.809
13	W→E	1	-11.397	13.103
14	E→W	1	-11.397	13.103
15	W→E	-1	-13.103	11.397
16	E→W	-1	-13.103	11.397
17	W→E	-3	-14.809	9.691
18	E→W	-3	-14.809	9.691
19	W→E	-4	-15.662	8.838
20	E→W	-4	-15.662	8.838
21	W→E	-2	-13.956	10.544
22	E→W	-2	-13.956	10.544
23	W→E	0	-12.250	12.250
24	E→W	0	-12.250	12.250
25	W→E	2	-10.544	13.956
26	E→W	2	-10.544	13.956
27	W→E	4	-8.838	15.662
28	E→W	4	-8.838	15.662
29	W→E	3	-9.691	14.809
30	E→W	3	-9.691	14.809
31	W→E	1	-11.397	13.103
32	E→W	1	-11.397	13.103
33	W→E	-1	-13.103	11.397
34	E→W	-1	-13.103	11.397

35	W→E	-3	-14.809	9.691
36	E→W	-3	-14.809	9.691
37	W→E	3	-9.691	14.809
38	E→W	3	-9.691	14.809
39	W→E	1	-11.397	13.103
40	E→W	1	-11.397	13.103
41	W→E	-1	-13.103	11.397
42	E→W	-1	-13.103	11.397
43	W→E	-3	-14.809	9.691
44	E→W	-3	-14.809	9.691
45	W→E	-2	-13.956	10.544
46	E→W	-2	-13.956	10.544
47	W→E	0	-12.250	12.250
48	E→W	0	-12.250	12.250
49	W→E	2	-10.544	13.956
50	E→W	2	-10.544	13.956
51	W→E	-2	-13.956	10.544
52	E→W	-2	-13.956	10.544
53	W→E	0	-12.250	12.250
54	E→W	0	-12.250	12.250
55	W→E	2	-10.544	13.956
56	E→W	2	-10.544	13.956
57	W→E	1	-11.397	13.103
58	E→W	1	-11.397	13.103
59	W→E	-1	-13.103	11.397
60	E→W	-1	-13.103	11.397
61	W→E	1	-11.397	13.103
62	E→W	1	-11.397	13.103
63	W→E	-1	-13.103	11.397
64	E→W	-1	-13.103	11.397
65	W→E	0	-12.250	12.250
66	E→W	0	-12.250	12.250

Table A2. Carriage positions for each pass for seating loads.

Pass Sequence No.	Direction	Carriage Centerline Location, ft.	
		North	South
1	W→E	-19.340	2.680
2	E→W	-19.340	2.680
3	W→E	-18.507	3.513
4	E→W	-18.507	3.513
5	W→E	-17.674	4.346
6	E→W	-17.674	4.346
7	W→E	-16.841	5.179
8	E→W	-16.841	5.179
9	W→E	-16.008	6.012
10	E→W	-16.008	6.012
11	W→E	-15.175	6.845
12	E→W	-15.175	6.845
13	W→E	-14.342	7.678
14	E→W	-14.342	7.678
15	W→E	-13.509	8.511
16	E→W	-13.509	8.511
17	W→E	-12.676	9.344
18	E→W	-12.676	9.344
19	W→E	-11.843	10.177
20	E→W	-11.843	10.177
21	W→E	-11.010	11.010
22	E→W	-11.010	11.010
23	W→E	-10.177	11.843
24	E→W	-10.177	11.843
25	W→E	-9.344	12.676
26	E→W	-9.344	12.676
27	W→E	-8.511	13.509
28	E→W	-8.511	13.509
29	W→E	-7.678	14.342
30	E→W	-7.678	14.342
31	W→E	-6.845	15.175
32	E→W	-6.845	15.175
33	W→E	-6.012	16.008
34	E→W	-6.012	16.008
35	W→E	-5.179	16.841
36	E→W	-5.179	16.841

37	W→E	-4.346	17.674
38	E→W	-4.346	17.674
39	W→E	-3.513	18.507
40	E→W	-3.513	18.507
41	W→E	-2.680	19.340
42	E→W	-2.680	19.340