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Elimination of Weight Restriction on Amtrak, NJ Transit, and Conrail Lines

REPORT
May 2012

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16. Abstract

New Jersey's freight railroad system facilitates movement of a large volume of freight, and provides relief from alternative transportation methods of freight transportation and their associated negative effects, such as congestion, pollution, roadway deterioration, and diminished safety. In New Jersey, freight rail cars often utilize a portion of the passenger rail network to complete their trips. However, much of this infrastructure currently operates with a weight limit of 263,000 pounds per rail car while the more common freight-rail car standard restriction is 286,000 pounds throughout the national freight network. Thus, an increase in the weight restriction to 286,000 pounds provides uniformity and allows freight-rail shippers and receivers to maintain the economic advantage freight rail provides.

Much of the infrastructure, such as bridges, was built prior to World War II, and the cost to build and maintain new infrastructure is extremely high. In this study, the research team investigated the impact of increasing rail car weight restrictions for bridges on passenger lines in New Jersey. The research approach adopted by the Rutgers Infrastructure Monitoring and Evaluation (RIME) Laboratory is aimed at evaluating the current load-carrying capacity of various types of bridges using AREMA Specifications, field testing, and finite element models. Based on load rating using FE modeling, it is found that sizable amount of repairs is needed for various structural elements of the Raritan Valley MP 31.15, the North Jersey Coast Line MP 31.15, and Bergen County Line MP 5.48 Bridges to improve their performance, fatigue life and maintain adequate safety margin. Based on AREMA's methods of evaluation, it was found that repairs are needed for all five bridges to maintain adequate safety margin for allowing 286-kips railcars. It is also recommended to utilize existing sensors to operate a long term monitoring system to evaluate the long term performance of the bridges and take advantage of in-place sensors. In addition to a Cost/Benefit analysis for a typical rail line with the HX Draw bridge, a tentative cost estimate of repairs for all five bridges is also included.

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EXECUTIVE SUMMARY

The overall growth in the economy and population in the United States led to a significant expansion of railroad traffic levels by the late 1990s. The freight railroad system facilitates a large volume of freight movement cost-effectively. It provides relief from alternative transportation methods, such as trucks and their associated negative impacts on roadway congestion, pollution, pavement and infrastructure condition, and safety.

In New Jersey, freight rail cars use a portion of the passenger rail system to complete their trips. Throughout the national freight network, weight limits have been moving from a previous standard of 263,000lbs to 286,000lbs. However, the passenger rail network has not been rated for 286,000lbs. Bridges in the passenger rail system were not designed based on the increased rail car weight. The impact of the increased rail car weights on these bridges require their evaluation prior to an increase in the weight restriction.

In this study, the impact of the increased rail car weight was investigated for bridges located on passenger rail lines in New Jersey. The research approach adopted by the research team (RT) is aimed at evaluating the current load-carrying capacity of several sample (or typical) bridges and providing recommendations for load rating, repair, and strengthening, to allow 286,000-lb rail car traffic on passenger lines.

Detailed literature review was conducted to find similar research and practices, followed by a review of inspection reports for candidate bridges for a 286,000-lb rating identified by the project partners. In the event that inspection reports were unavailable, or detailed information was insufficient, current bridge conditions and actual dimensions of the bridges were evaluated from field inspections. Based on the input from New Jersey Transit (NJ Transit) and field inspections, five bridges on NJ Transit's rail lines were selected and load-rated based on the current American Railway Engineering and Maintenance-of-Way Association (AREMA) Specifications. Bridges with various structural systems and material types were selected. Finite element models were developed and validated using data gathered from field tests for more accurate assessment of the bridges and to help develop a more accurate methodology for evaluating and load-rating railroad bridges. The selected bridges were instrumented and tested under live loads (moving rail cars). Finally, recommendations for load rating, maintenance, repair, and rehabilitation of the bridges, including cost estimates, were provided for safe operation of the bridges. The recommendations will be applicable for other similar railroad bridges that support rail cars with the increased standard weight.

Briefly, this project addresses problems with the existing railroad bridges under the increased 286-kips rail car loading. Through this research, the detailed structural evaluation and load rating was performed using AREMA approach, field-testing, and FE modeling. Furthermore, the RT provided guidelines for maintaining and load-rating existing railroad bridges, as well as the cost-effective analysis of this change in the freight weight limits. Based on the load rating using FE modeling, it was found that a sizable amount of repair is needed for various structural elements for the Raritan Valley MP 31.15, North Jersey Coast Line MP 31.15, and Bergen County Line MP 5.48 to improve their performance, extend fatigue life of the bridge, and maintain adequate safety margin. Based on AREMA's methods of evaluation, it was also found that repairs are needed for all five bridges to maintain adequate safety margin based on load rating using simple beam analysis. Moreover, the fatigue analysis performed in this study indicated that the remaining fatigue life of the bridges would be reduced by a percentage of 35-50% minimum, if the 286-kips freight railcar were utilized. Thus, in order to evaluate the long-term performance of these bridges and take advantage of in-place sensors, it recommended that further data collection and long term structural monitoring before and during operation of 286-kips railcars be performed.

INTRODUCTION

By the late 1990s, economic and population growth resulted in significant freight movement. It is expected that rail freight traffic will grow sharply for the next 20 years. Therefore, substantial demand will be put on the already heavily-used railroad system.[(1)] The freight railroad system enables cost-effective movement of a large volume of freight, and is important because the alternative transportation methods, vehicles and trucks, cause concerns about congestion, air quality, and safety.

However, the cost to build and maintain infrastructure and equipment is very high, and it is very difficult to make long-term investment in railroad infrastructure.[(2)] Additionally, many railroad bridges were built before World War II and are approaching the end of their assumed design service life, which creates additional concerns.

In New Jersey, freight rail cars use portions of the passenger rail system to reach their destinations, sharing lines with NJ Transit commuter rail service. An increase of maximum rail car weight from 263,000lbs to 286,000lbs raises concerns for the passenger rail system, since its bridges were not designed for 286,000-lb cars.

In Wisconsin, railroad bridges were evaluated to determine the impact of rail car weight increase.[(3)] Field investigations and load ratings were conducted to determine the conditions of the bridges and provide recommendations for safe operation of the 286-kips rail cars on the bridges. A total of 26 sample bridges were inspected and load-rated. Most of the bridges were timber bridges, but three steel bridges and one concrete bridge were also investigated in the study. In the load rating, a Cooper E80 load and a 286-kips rail car load were used to determine the maximum load effects and stresses. The configuration of the Cooper E80 load and 286-kips rail car load are shown in Figure 1 and Figure 2, respectively. As required by AREMA, the bridges were load-rated with two levels of rating conditions, normal and maximum.

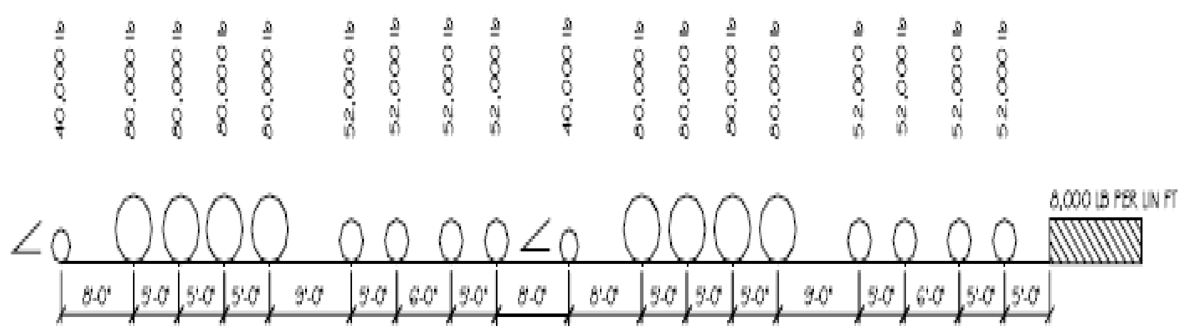


Figure 1. Cooper E loading[(3)]

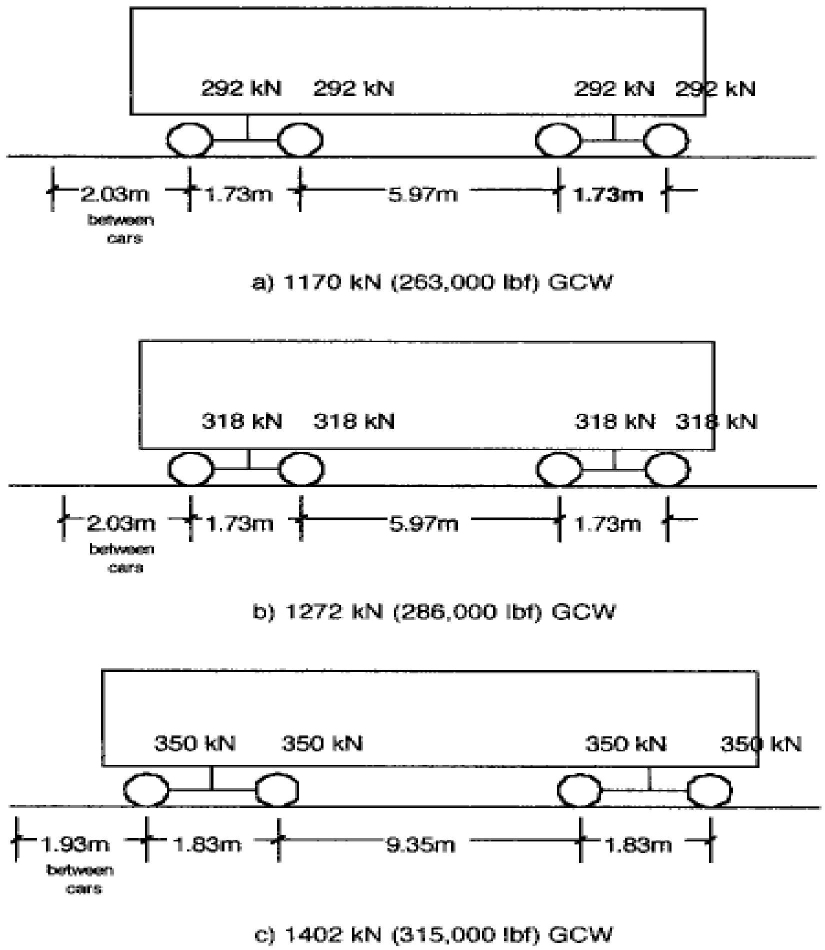


Figure 2. Rail car loading configurations[(4)]

The analysis results showed that many of the timber bridges were not able to carry the 286-kips rail car, while load ratings for the steel and concrete bridges indicated that they could adequately carry the 286-kips rail car. The study also showed that the timber bridges investigated were in poor to fair condition, and the concrete and steel bridges were in moderate to good condition. Based on the field investigation results, the report also provided estimates for the remaining service life, approximate maintenance costs, and recommendations for repair.[(3)]

A similar study was conducted by Leighty III et al. to evaluate the effects of rail car weight increase on the Pennsylvania railway network bridges.[(4)] There are over 2,000 bridges on Pennsylvania short-line railroads (SLRRs), and 1,174 bridges were considered in their study. A total of 25 bridges were selected for structurally evaluation. Leighty III et al. investigated the load-carrying capacity of the selected bridges and estimated repair cost for the bridges that were not able to carry the increased rail car weight.[(4)] Field investigations were also conducted and the railroad bridges were load-rated using the Cooper E loading and various other weight rail cars loading (263 kips, 286 kips, and 315kips, see Figure 2). Many bridges were not able to carry 286-kips rail car loading. However, some of the bridges that underwent past repairs or strengthening were able to carry the higher 315-kip rail car loading. For the bridges that did not meet the load rating, strengthening methods were developed, and strengthening costs were also obtained from experienced bridge engineers.

Many previous studies emphasized the importance of load tests of existing bridges. Fryba and Pirner presented various methods to evaluate conditions of existing bridges.[(5)] They provided criteria for the results of static load tests, including calculated deflection versus measured deflection, permanent deflection, and crack width under heavy vehicle loading. The bridge conditions can also be evaluated by studying the dynamic impact factor and natural frequencies from dynamic tests. The steel bridge stress-monitoring method is also presented in Fryba and Pirner.[(5)] The long-term data of the stress ranges in structural components, as shown in Figure 3, can be used to estimate their fatigue life and determine inspection intervals. Finally, they provided a modal analysis method to detect damages in the structures.

James studied the load effects on railroad bridges with short to medium spans in Sweden.[(6)] Field data were obtained with variables including train speed, axle loads, and axle spacing. Bridge models were developed and the load effects were analyzed by probabilistic models. The bridges were simplified to two-dimensional models and the dynamic effects of moving rail cars were considered in the analyses. With the model, reliability analyses with the generalized extreme value distribution and peaks-over-threshold method were conducted to determine the availability of raising allowable axle loads on existing railroad bridges. Based on the studies, it was concluded that an increase of axle load to 25 tons would be acceptable for the existing bridges designed based on Load Model A used in the 1940s.[(6)]

In the study conducted by Leighty III et al., the estimated costs for the higher rail car operation, based on the 25-sample bridge study, were extrapolated for the entire SLRR bridge population in Pennsylvania to conduct the statewide cost evaluation for the rehabilitation.[(4)] To achieve the objective, they considered the costs of bridge inspections, screening analysis, detailed structural analysis, detailed strengthening design, and construction costs. It was estimated that the total cost required to upgrade the entire SLRRs in Pennsylvania was over $8,000,000.

Chebrolu et al. developed a long-term health-monitoring system using a wireless sensor network for railroad bridges in rural area.[(7)] With health monitoring of the bridges, the data can be used to evaluate current conditions of the bridges and track the deterioration over time. Compared to previous work, they used an event-detection mechanism to trigger data collection (see Figure 4). The adopted health-monitoring method minimized maintenance of the installed systems.

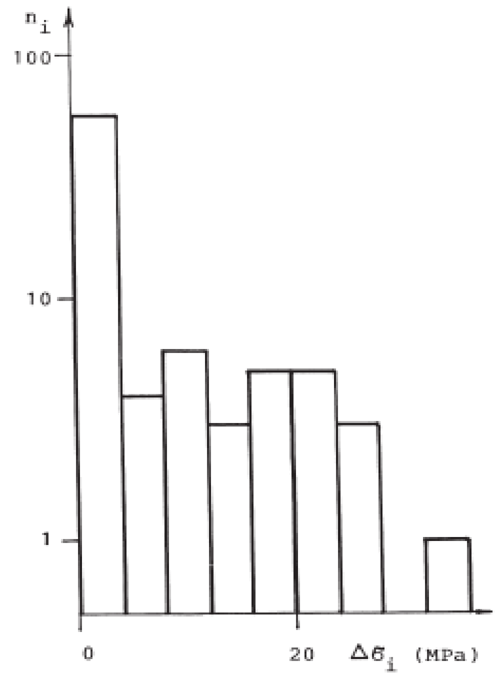
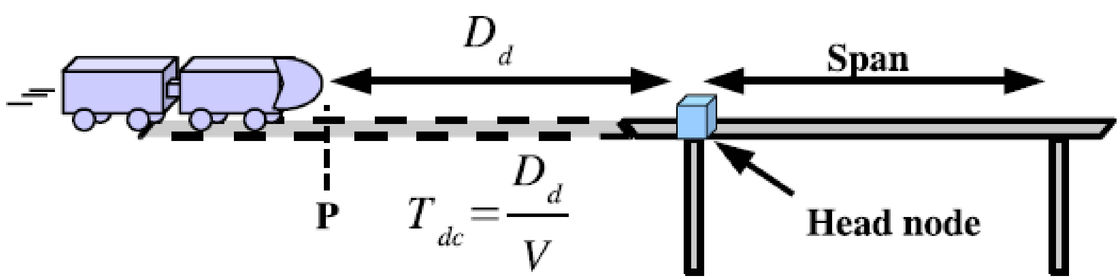


Figure 3. Sample of stress spectrum in steel railroad bridge.[5]
($\Delta\sigma$ = Stress range, n = Number of cycles)



Note: D_d : Distance from span at which the train can be detected V : Train speed

$T_{dc} = D_d / V$: The maximum time between train detection and start of data collection

Figure 4. Detecting an oncoming train[7]

The increase of rail car loading on the existing steel railroad bridges may increase the number of high-stress cycles, resulting in fatigue damage in the bridges. However, fatigue strength of existing steel bridges is hard to predict because of many unknown factors. A method to predict remaining fatigue strength of existing riveted railway bridges was developed by Tobias and Foutch.[8] Because of the uncertainties of fatigue strengths, bridge responses, and loadings, the reliability theory was used in the study. Based on the method, they concluded that the bridges with smooth rivet holes and tight fasteners have longer fatigue life than the bridges without smooth rivet holes and tight

clamping force. However, the remaining life of short-span bridges may decrease significantly with increasing rail car weight.
Previous research studies presented above show various methods for evaluating existing railway bridges, including reliability theories and wireless health monitoring, as well as conventional field investigations and load rating. To evaluate the impact of the 286-kips rail cars on bridges on NJ Transit lines, this research study adopted state-of-the-art methods and combined them with the experience of the RIME RT obtained from previous research projects.

Objectives

The main objective of this study is to evaluate current conditions of selected railroad bridges, and load-rate them according to AREMA provisions, to evaluate whether to allow travel of 286-kips rail cars.(9) Furthermore, field tests and detailed finite element analysis were conducted for more accurate condition evaluation of the bridges. Based on the study of the selected railway bridges, general guidelines for bridge repair and strengthening to accommodate 286-kips rail car loads are also provided in this study.

LITERATURE REVIEW

In New Jersey, freight rail cars travel over many passenger rail lines. A recent increase in standard rail car weight from 263,000 lb to 286,000 lb raised concerns about their usage in passenger rail systems, since bridges on these lines were not designed for the increased rail car weight. Investigation and evaluation of these bridges for 286,000-lb loading was deemed necessary in order to continue to use these lines for freight traffic and explore the possibility of allowing the standard 286,000-lb rail car.

As a first step in the project, previous research studies and practices were reviewed. Similar studies were conducted by other transportation agencies.(3) The Wisconsin Department of Transportation (Wisconsin DOT) and the Pennsylvania Department of Transportation (Pennsylvania DOT) recognized the problem of the load-carrying capacities of existing railroad bridges, and conducted bridge inspection, load rating, and structural evaluations for the existing railroad bridges.

Field tests of the railroad bridges were conducted in many research studies.(10,11,12) Due to deterioration, complex geometry, unexpected restraints, effects of nonstructural elements, repair, and modifications, the behavior of the railroad bridge under train loading can be different from the intended behavior at the time of design and construction. Field tests sometimes provide engineers with better understanding of the bridge behavior and load rating and a valuable way of verifying the results obtained from mathematical analysis. Information regarding test methods, instrumentation, and test setup was gathered in this study to obtain better data from the possible bridge testing.

Because of financial and practical limits, field tests cannot be conducted for all bridges. Instead, finite element analysis can be adopted for accurate load rating of bridges. A higher degree of confidence in the results of finite element analysis can be achieved by verifying results from field-testing and calibrating models as appropriate. Previous research studies on finite element modeling of railroad bridges were reviewed to build accurate bridge models.

Brief descriptions of load rating and strengthening methods of concrete, steel, and timber bridges are also presented in this study. AREMA also provides methods for repair and strengthening of existing railroad bridges.[9]

Bridge Load Rating Conducted by Other DOTs

Wisconsin DOT

A similar study was conducted on Wisconsin's railroad system to determine the impact of 286-kips rail cars. The scope of the project covered evaluating current conditions, determining load-carrying capacities, and making repair and retrofit recommendations based on the investigations.

In the study, 26 sample bridges were selected on two rail lines operated by Wisconsin & Southern Railroad Co. The selected bridges consisted of steel bridges, concrete bridges, timber bridges, and combined timber-steel bridges. The sample bridges were built between 1900 and 1965.

The evaluation of the 26 sample bridges in Wisconsin showed that a sizable amount of maintenance and repair was required for the bridges to support 286-kips freight cars. The study estimated the repair and strengthening cost of all railroad bridges in Wisconsin owned by the Wisconsin & Southern Co. over the next five years, which was about $25 million. For the sample bridges, about $3 million was required to allow 286-kips freight cars.

Pennsylvania DOT

Pennsylvania State University, sponsored by the Pennsylvania DOT, investigated the impact of higher rail car weight on the load-carrying capacity of SLRRs. Out of 2,000 bridges located on Pennsylvania short-line railroads, 1,174 bridges were under consideration, and 25 sample bridges were selected and evaluated based on field test results and American Railway Engineering Association (AREA) specifications.[4] Through a combination of mail surveys, telephone interviews, and on-site visits, data was gathered from each bridge. The data gathered included milepost location, bridge type, length, construction material, construction data, bridge width, gross car weight (GCW) capacity, date inspected, description of physical condition, and availability of bridge plans.

For the selected 25 bridges, field inspections were conducted to evaluate current conditions of the bridge that can be used for the load rating. This includes section loss of structural members, unrecorded repair, and damage. The field inspection results were then used for load rating. The sample bridges were evaluated for five different loadings, which are Cooper loading, alternative live load, 263-kip rail car, 286-kips rail car, and 315-kip rail car. The loadings were applied with standard impact factor without a reduction corresponding to speed.

In the study, evaluation results were reported as the percentage ratio of allowable resistance to applied load. The results are listed in Table 1. As shown in the table, five bridges out of the 25 sample bridges cannot carry the 286-kips rail car loading. Of the sample bridges, 12 bridges meet E80 loading criteria. Note that many bridges can carry 315-kip rail car loading.

Table 1 - Capacity–load ration in percentage[(4)]

Bridge no.	*Spans*	*Length (m)*	*Type*	*Fig 1 (a) 263 k (%)*	*Fig 1 (b) 286 k (%)*	*Fig 1 (c) 315 k (%)*	*Cooper E*	*AREA Alt E*
1	4	85.40	DPG	135	126	123	69.9	76.7
2	6	91.04	DPG	155	144	138	80.0	87.5
3	1	27.91	DPG	135	126	141	73.8	101.3
4*	**1**	**14.64**	**TPG**	**99**	**92**	**87**	**56.1**	**48.1**
5	1	12.20	DPG	205	189	173	119.1	106.6
6	1	16.78	DPG	171	159	149	91.5	95.9
7	1	24.40	DPG	164	153	157	91.0	118.1
8*	**1**	**7.32**	**TPG**	**89**	**83**	**77**	**49.1**	**42.3**
9	1	10.22	SST	247	240	233	326.6	286.7
10	1	13.72	TPG	137	130	122	85.5	81.1
11	1	9.46	DPG	116	107	102	67.2	58.3
12	1	21.96	DPG	157	146	157	93.2	111.1
13	1	28.98	DPG	158	148	162	92.5	134.7
14	2	46.05	DPG	213	199	215	119.5	150.7
15	1	30.50	DTR	114	107	121	66.0	95.0
16	1	46.36	TTR	110	103	105	62.7	89.0
17	1	3.05	MAR	133	132	132	363.5	2253.8
18*	**1**	**4.88**	**MAR**	**40**	**35**	**35**	**20.0**	**18.0**
19*	**1**	**2.75**	**TST**	**64**	**59**	**55**	**39.5**	**31.5**
20	1	2.14	CSB	122	113	108	73.9	59.1
21	1	13.73	DPG	153	144	138	92.4	93.1
22	18	393.50	DPG	131	126	129	91.8	126.1
23	4	55.21	DPG	146	136	136	79.4	86.7
24*	**1**	**45.90**	**TTR**	**101**	**93**	**84**	**64.2**	**66.5**
25	2	36.60	DPG	109	102	113	56.3	73.6

For bridges that could not carry the heavier 286-kips rail car and 315-kip rail car, cost-effective methods were developed to strengthen the bridges. The strengthening scheme included post-tensioning floorbeam, alleviating soil pressure on the wall of the arch bridge, attaching steel channels to timber stringers, replacing deteriorated timber members, and adding ties to steel truss members.

Field Tests on Railroad Bridges

<u>Timber Trestle Bridges (Colorado State University)</u>

Colorado State University and the Association of American Railroads (AAR) conducted field tests of a timber-trestle railroad bridge to determine the effects of additional stringers on the stiffness of the bridge. A three-span timber bridge tested in the study is shown in Figure 5. The bridge is about 40 ft long, and main components were made of creosote-treated Douglas fir timbers.

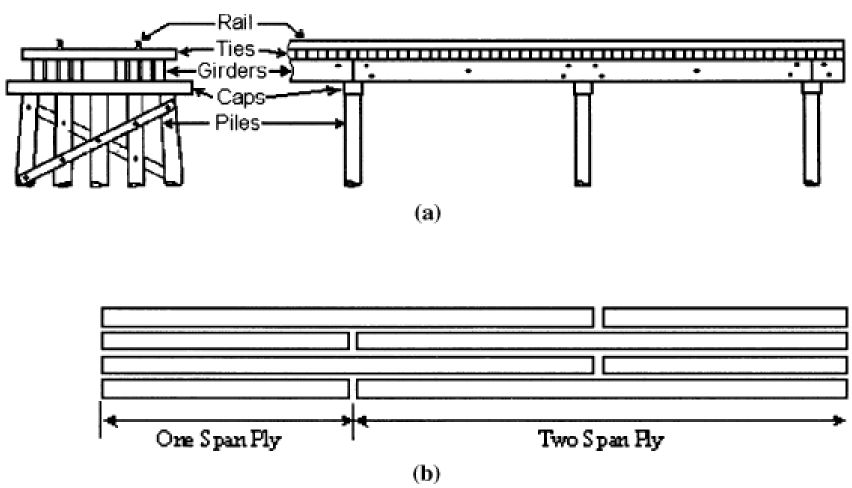


Figure 5. Open-deck timber bridge: *(a)* schematic drawing; *(b)* stringer layout

The bridge was instrumented with various sensors, including displacement transducers, extensometers, optical surveying equipment, and accelerometers. The linear variable differential transducers (LVDTs) were installed to measure relative displacement between components and vertical displacement. Vertical displacements of the bridge during the static tests were also measured with the optical survey equipment. Accelerometers were used for the moving-load testing.

Loading was applied to the bridge using the AAR’s track-loading vehicle (TLV), as shown in Figure 6. The TLV is able to apply concentrated loading to railroad track using hydraulic actuators. By moving the TLV at various locations, the bridge was tested under static loading. Moving-load tests were also conducted while the test train passed over the bridge. The test train consists of a locomotive, instrumentation car, and TLV. In the study, sinusoidal loading using the TLV actuator was also applied to the bridge.

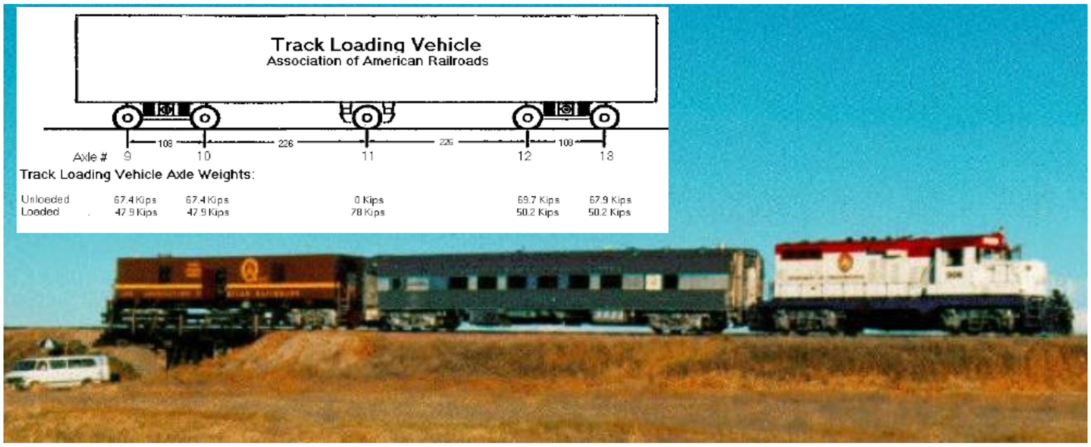


Figure 6. Test train[10]

Built-up Steel Girder Bridge (Rutgers University)

Nassif et al. tested two steel girder bridges located in New York. The bridges contain three spans with simply supported girders. The thru-girders are riveted built-up girders, and the overall length of the bridge is about 60 ft (Figure 7).[11] The field tests were conducted to evaluate stresses and deflections of the bridges under passenger train loading and compared with allowable stresses according to the current provisions.

Strain transducers and LVDTs were installed to measure strains and displacements, respectively. A portable data acquisition system was used to obtain data under the train loading. The strain transducers and LVDTs were installed at the south span because the span easier to access than the other spans. Installed sensors are shown in Figure 8. The strain transducers were installed in the steel flange with C-clamps or to a custom-made steel plate attached to the bottom of the trough. The LVDTs were installed on a temporary platform.

Figure 7. Three simple spans bridge used in the field tests [11]

Figure 8. Sensors used to measure strains and deflection(11)

Static tests were conducted by positioning a rail car at predetermined locations at night. A field engineer from Metro-North communicated with an onboard engineer to position the train at the locations (Figure 9). The train was stopped at each location for 2 to 5 minutes to obtain static test results. Typical deflection results from the static loading are shown in Figure 10. Dynamic live-load tests were also conducted under multiple train passages with known axle weights. These dynamic tests were conducted to evaluate the dynamic impact on the bridges. The trains were passed over the bridge at 10 mph and 70 mph.

Figure 9. Train positioned for static test(11)

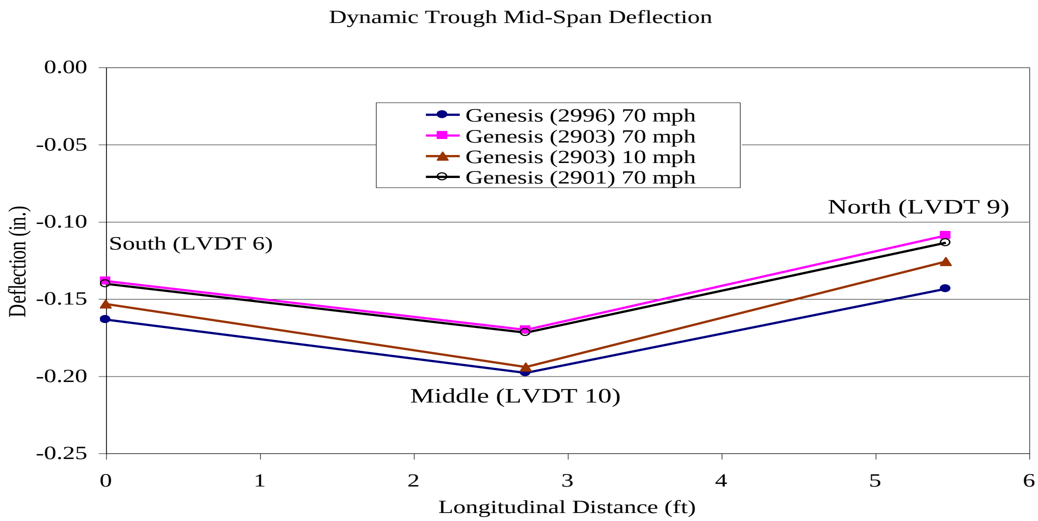


Figure 10. Typical deflection profile in trough under train loading(11)

Built-up Steel Girder Bridge (University of Delaware)

Chajes et al. at the University of Delaware conducted load tests and in-service monitoring of a steel-girder railroad bridge on NJ Transit line. The bridge is about 45-ft long, with simply supported girders (Figure 11).(12) The bridge has two tracks, but only one track is used for service. The low load rating of the bridge necessitated a low-speed restriction. To evaluate the steel-girder bridge under the train loading, strain transducers were installed on the bridge.

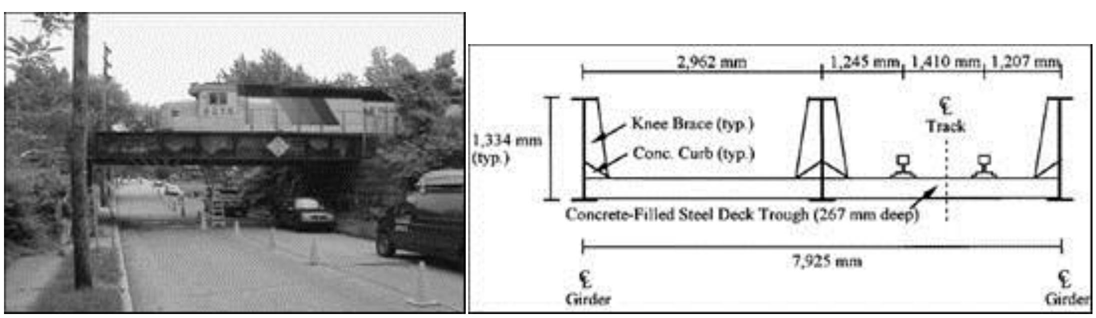


Figure 11. Tested bridge(12)

The load test was conducted using the regularly scheduled transit train without interrupting train service, and locomotive weight was provided by NJ Transit. Typical strain-time history data measured under the moving passenger train is shown in Figure 12. An in-service monitoring system was installed in the bridge after the load test, and the stresses in the structural components were measured for a week. The system automatically records time, peak strain, and strain transducer number when the strains in the sensors are higher than a pre-specified strain limit. The test results showed that the measured stresses were 15 percent of the computed stresses in the load rating, indicating possible increase of current load rating of the bridge.

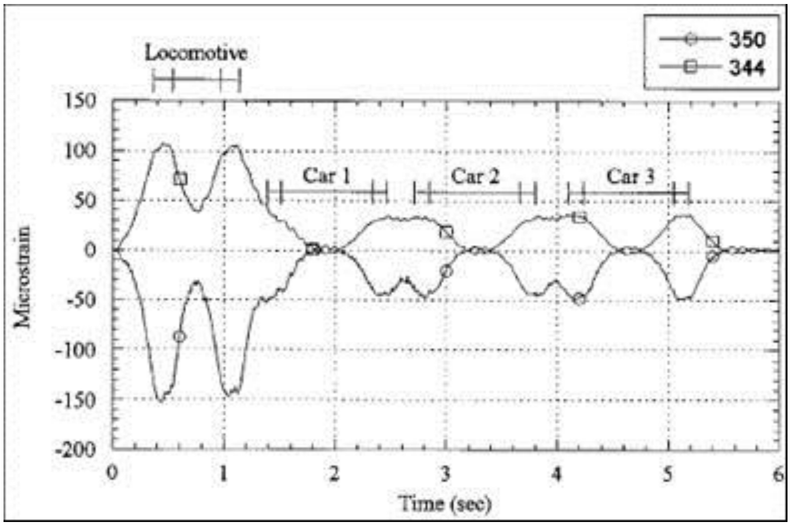
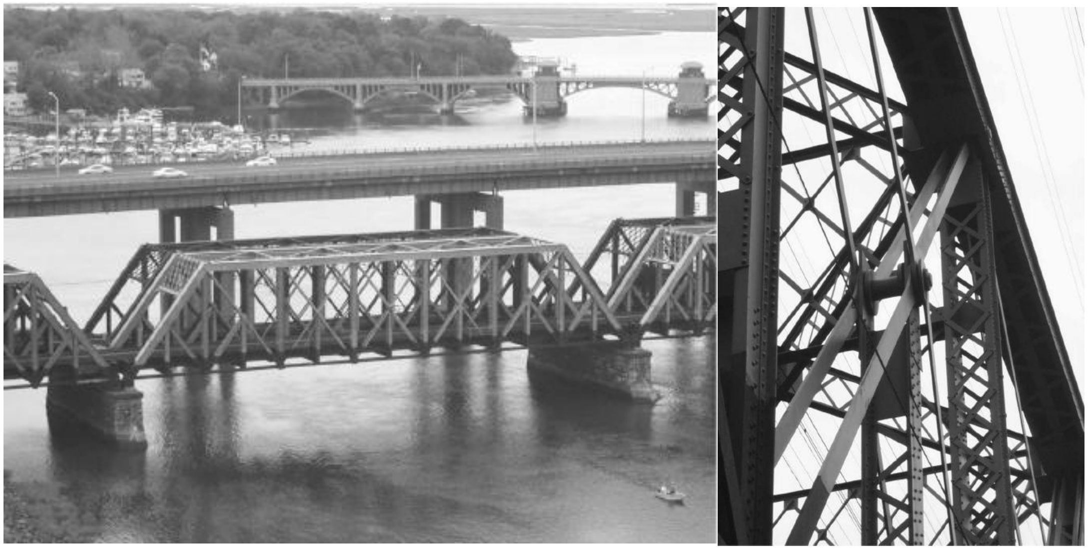


Figure 12. Typical strain time history data[12]

Built-up Truss Bridge (University of Connecticut)

DelGrego et al. conducted tests on a truss railroad bridge with built-up section members.[13] Field monitoring under regularly scheduled train loading were conducted to evaluate the structural behavior and live load distribution in the bridge. The bridge tested by the team from the University of Connecticut was one of the typical large-truss bridge structures that were constructed with eyebars, small angles, channels, plates, lacing, and bars. The connections were made with large pins and rivets. The bridge was monitored because of the lateral movement of the mid-depth pins on the bridge. The experimental data provided an opportunity to compare the bridge behavior under the train loading with the expected behavior in the original design that was conducted more than 100 years ago.

The bridge tested has seven spans, and the tested span is 210-ft long (Figure 13 (a)). A total of 372 weldable strain transducers were placed on different truss members, primarily to tension members. The main interests of the tests were the load distribution in diagonal members, load sharing in multiple eyebars (Figure 13 *(b)*), and the influence of floor beam (FB) rotation on the adjacent truss members.



(a) Tested span *(b)* Multiple eyebars

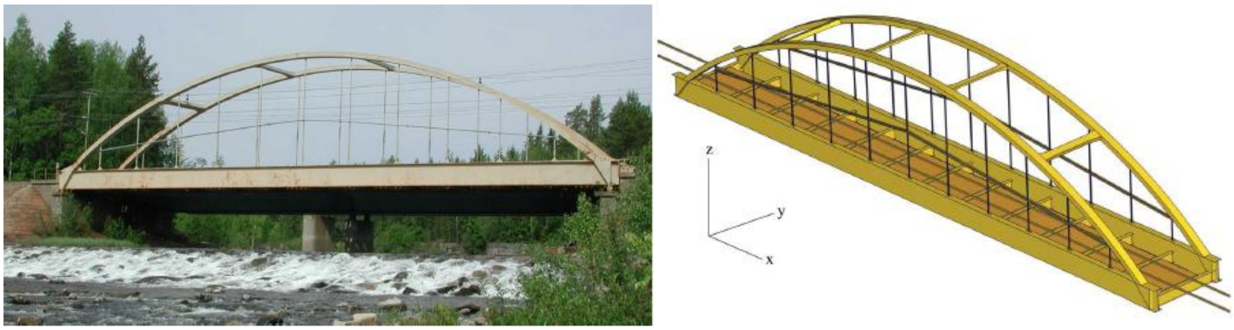
Figure 13. Tested railroad truss bridge[13]

Tests were conducted and data were collected for 16 different trains. In the study, DelGrego et al. emphasized a significant influence of aging on the load-carrying ability of the century-old truss.[13] The difference in behavior from the design assumption was also noted in the study.

Finite Element Modeling

Railway bridges have relatively short spans compared to bridges designed for highways and truck traffic. Many railroad bridges have been in service for a long time and are composed of timbers, built-up truss, and riveted built-up sections. Because of the simplicity of structural systems, simple frame analysis was adopted in many previous research studies.[See references 10,11, 12 and 14] Detailed finite element analysis for railroad bridges can be found in studies on the dynamic interaction between rail cars and tracks.[15,16]

Malm and Andersson investigated the dynamic effects of train passages on a tied-arch bridge.[17] The bridge investigated is 147.6-ft long and used for both passenger and freight train traffic. The bridge consists of two hollow arches without ballast. The finite element (FE) model shown in Figure 14 was developed using the general-purpose FE program, ABAQUS. The developed model was used to compare the field test results with the simulation results and to better understand the bridge behavior under moving-train loading. Dynamic characteristics and the structural behavior of the bridge were investigated with the model, as well as with the field tests.



(a) Investigated bridge *(b)* Bridge FE model

Figure 14. Tied-arch bridge[17]

At the University of Porto, Portugal, an arch bridge used for urban road traffic was evaluated for possible use of the light rail.[18] A detailed FE model was developed to evaluate the interaction between the bridge and the trains and dynamic amplification factors for the moving train loads. The model was built using the beam element, and the developed model was validated with field test results. After the validation, numerical simulations were conducted to evaluate structural behaviors under the rail traffic.

Song et al. investigated the interaction between high-speed trains and bridges, and the analysis results from the model were compared with test results conducted on the bridge.[15] The deck of the bridge was modeled with the shell element, and the track structures were modeled with the beam element. Spring elements were used to model ballast (Figure 15). In the study, a high-speed train model was also developed to investigate the interaction between the train and the bridge.

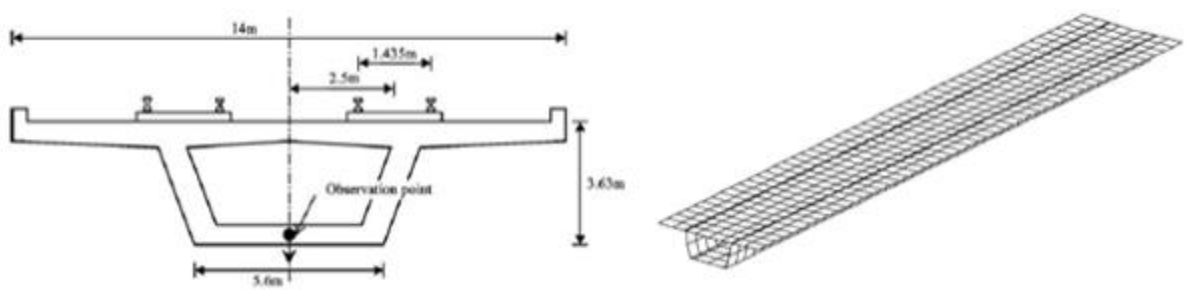


Figure 15. Finite element model of bridge for high-speed train

Summary

The needs for condition evaluation of existing railroad bridges have been identified by other transportation agencies. Wisconsin DOT and Pennsylvania DOT conducted research on the existing bridges to allow the 286-kips rail cars. The research results showed that many timber trestle bridges do not satisfy the load rating for the increased

rail car weight. Field investigation of the bridges indicated significant maintenance needs over several years.
Moreover, field tests of railroad bridges have been conducted by many research agencies to identify the behavior of bridges and to evaluate stresses in the structural components under moving train loading. Bridge strains, deflections, and accelerations were measured in many studies. Many field tests were conducted under normally scheduled train loading, while some tests were conducted by stopping trains at predetermined locations. To further evaluate the bridge behavior, FE analysis programs were used in many studies. Field test results were also used to validate the developed models, and parametric studies were conducted using the validated models.

Due to the large number of railroad bridges, it is not practically or financially possible to replace many structurally deficient bridges in a short period of time. Thus, studies have been conducted to develop efficient repair and strengthening methods for existing bridges. Approaches similar to those developed by other researchers were also used for existing bridges in New Jersey.

SELECTED BRIDGES FOR ANALYSIS

Five NJ Transit passenger rail line bridges were evaluated for the 286-kips rail car loading. Figure 16 shows five bridges identified by project partners NJ Transit and the New Jersey Department of Transportation's (NJDOT's) 286-kips Task Force that were selected based on freight rail car traffic use of these bridges. These bridges are also selected for future inclusion in New Jersey's 286-kips rail network. The Rutgers Team (RT) reviewed inspection reports provided by NJ Transit and NJDOT and constructed FE models of these bridges. The field testing was also conducted for all five bridges and provided a methodology for evaluating and load-rating these bridges using three approaches: 1) AREMA provisions, 2) field test data, and 3) detailed FE models. General information about the five bridges and associated testing is listed in Table 2. Each bridge is briefly described in the following section.

The five bridges are as follows:

Bridge I: Main Line MP 15.95

Bridge II: Main Line MP 15.14

Bridge III: Bergen County Line MP 5.48 (HX Draw)

Bridge IV: Raritan Valley Line MP 31.15

Bridge V: North Jersey Coast Line MP 0.39 (River Draw)

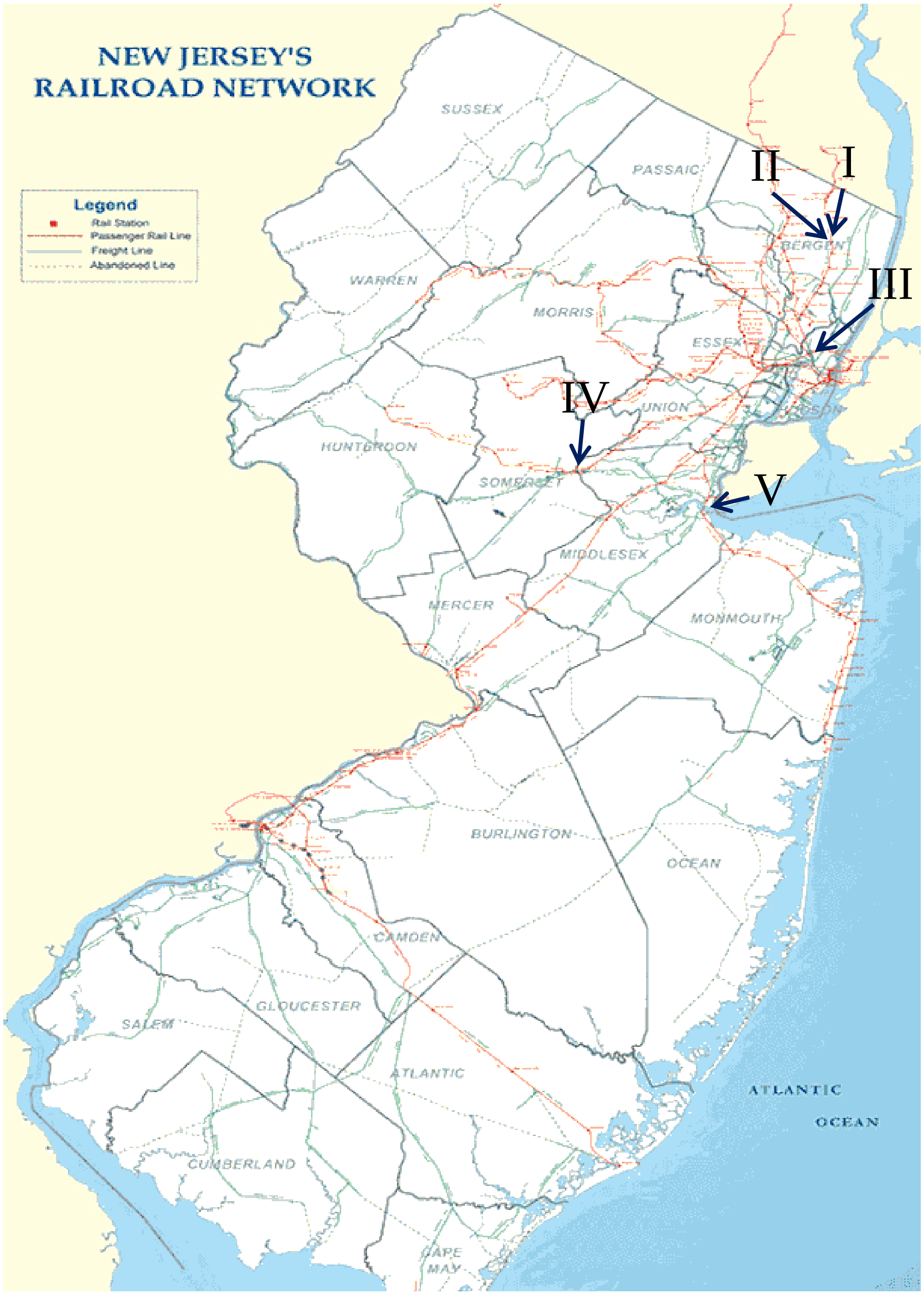


Figure 16. Selected bridges for testing and modeling

Table 2 - General description of selected five bridges

Bridge	Type of bridge	Bridge length	Number of tracks	Test span	Test variables	Test train types	Total test runs
Main Line MP 15.95	Steel-plate girders with floorbeams and ballast concrete deck	3 spans 74 ft	2 active tracks	Span 2	Strain deflection velocity	GP40-PH-2B, PL-42AC, GP40PH-20, GP40FH-2M	4
Main Line MP 15.14	Steel plate through girder and deck girder with floorbeams	7 spans 344 ft	2 active tracks	Spans 1&2	Strain, deflection velocity	PL42-AC, F40PH-2CAT, GP40FH-2M	3
Bergen County Line MP 5.48	One-truss steel bascule span, one steel through truss tower span, and 15 steel plate girder spans	17 spans 1,095 ft	2 active tracks	Spans 2, 3, 9,& 12	Strain	PL-42AC (286-kips freight rail car)	30
Raritan Valley Line MP 31.15	Steel plate through girders, open-hearth steel	4 spans 163 ft	2 active tracks	Spans 2&3	Strain, deflection velocity	PL-42AC	14
North Jersey Coast Line MP 0.39	Steel-truss swing span flanked by 28 steel-deck girder spans	30 spans 2,919 ft	2 active tracks	Spans 20& 26	Strain	ALP-46, ALP-46A	6

Bridge A: The Main Line MP 15.95 Bridge over Broadway

The Main Line MP 15.95 Bridge is a three-span bridge with a total span length of 74 ft over Broadway in Paterson, New Jersey. It carries two active tracks with a ballast deck. Based on the latest inspection report, the controlling member is Girder 2, with a normal rating of E52.[19] An elevation view of the bridge and its location on Google map is shown in Figure 17.

Figure 17. Elevation view of the main span and the controlling member for the load rating (center girder), view from Google Maps

Bridge B: The Main Line MP 15.14 Bridge over Straight Street and 21st Street

The Main Line MP 15.14 Bridge is a seven-span bridge with a total span length of 344 ft over Straight Street and 21st Street. This bridge was built in 1905, which makes it more than a century old. Based on the latest inspection report, the controlling member is Floor Beam 20 at Span 1, with the normal rating of E35 using as-inspected section properties.[20] Figure 18 shows the plan view of the bridge.

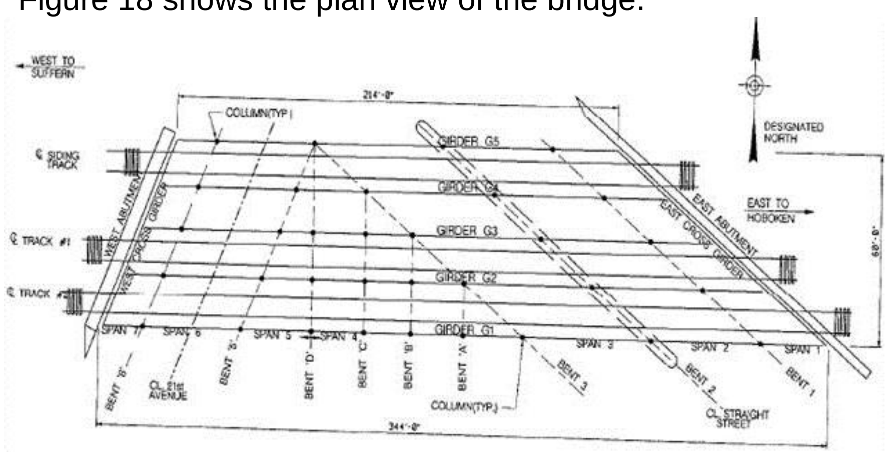


Figure 18. Plan view of the *Main Line MP 15.14* superstructure from Inspection Report Cycle 4[20]

*Bridge C: The Bergen County Line MP 5.48 (HX Draw**)** Bridge over Hackensack River*

The HX Draw Bridge is a 17-span bridge with a total length of 1,095 floated over the Hackensack River. This structure carries two active tracks over the Hackensack River between Secaucus, Hudson County, and East Rutherford, Bergen County, New Jersey. Based on the Inspection Report Cycle 4, the controlling member is the north girder below Track 2 at Span 3.[(21)] A general view of the bridge is shown in Figure 19.

Figure 19. General view of Span 3, Span 9 and Span 12 of the *Bergen County Line MP 5.48 (HX Draw**)** Bridge over Hackensack River* Bridge from Inspection Report Cycle 4[(21)]

Bridge D: The Raritan Valley Line MP 31.15 over Middle Brook Bridge

The Raritan Valley Line MP 31.15 Bridge is a four-span bridge with a total span length of 164.5 ft over Middle Brook. The bridge was built in 1902 with a superstructure fabricated with open-hearth steel. Based on the latest inspection report, the controlling member is the north girder below Track 2 at Span 2.[(22)] An elevation view of the bridge and the location of the critical span from the load-rating calculations are shown in Figure 20.

Figure 20. General view of the bridge from Inspection Report Cycle 4[(22)]

Bridge E: The North Jersey Coast Line MP 0.39 over Raritan River (River Draw) Bridge

The River Draw Bridge is a steel-truss swing bridge with 28 deck-girder approach spans having a total length of 2,918 ft over Raritan River. The bridge was erected in 1906 and carries two electrified tracks between Perth Amboy and South Amboy, New Jersey. Based on the Inspection Report Cycle 4, the controlling member is the 88-ft approach span girder and the swing-span-end floor-beam connection.[23] Figure 21 shows the elevation view of the bridge.

Figure 21. North elevation of east approach Span 1 to 18 of the River Draw Bridge taken from Inspection Report Cycle 4[23]

Typical 286-kips Rail Cars

The objective of this study is to evaluate the live load effects of typical 286-kips rail cars on the selected bridges. In addition to the 286-kips rail car used in the project, five different 286-kips rail cars were also investigated to evaluate the effects of different rail-car configurations. Table 3 shows diagrams of various 286-kips rail cars. Among these 286 rail cars, Numbers 2 through 4 were taken from the web page of Freight Car America, Inc. The reasoning behind selecting those three rail cars is that rail cars with closer axle spacing provide more conservative values. Rail cars number 2, 3, and 4 have the shortest axle distances among rail cars available in the Freight Car America catalogue.

Rail car Number 5 is taken from a study sponsored by the Wisconsin DOT titled "Impact of Railcar Weight Change on Bridges of the State of Wisconsin Owned Railroad System." Rail Car Number 6 is taken from a study that was performed by Horney and presented in the American Short Line and Regional Railroad Association's AREMA Conference in 2003, in Chicago, Illinois.[24] This rail car represents a model rail car used to develop a program that provides a consistent methodology for evaluating the timber bridge inventory. It is noted here that although NJ Transit used Rail Car Number 6 in their studies, Rail Car Number 1 was selected for FE analysis in this study since Type I Rail cars are the shortest and produce the largest live load effects. Table 4 shows a list of various cars that have been tested in this study, including the locomotives and 286-kips rail cars.

Table 3 - 286-kips rail car diagrams

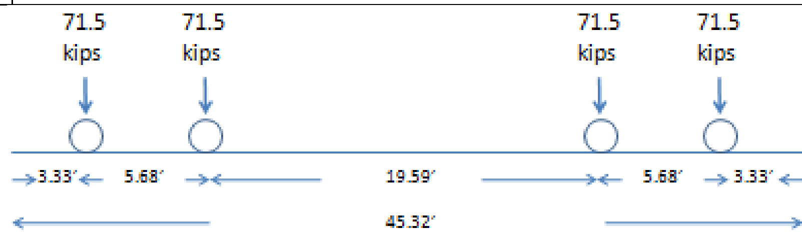
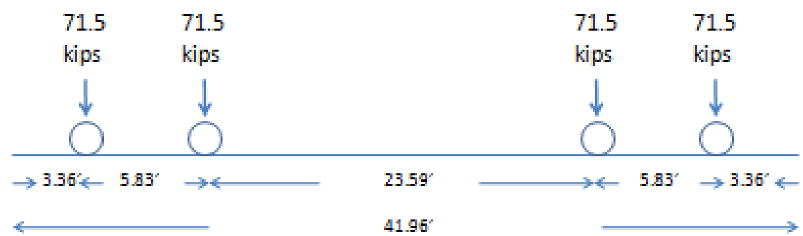
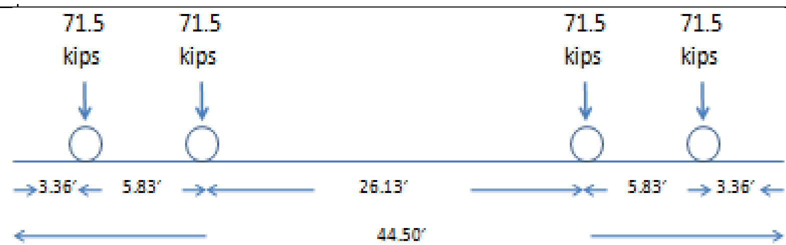
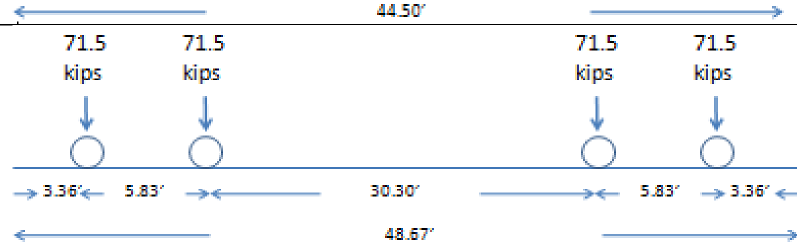
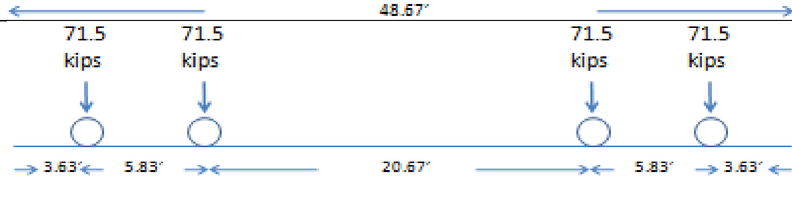
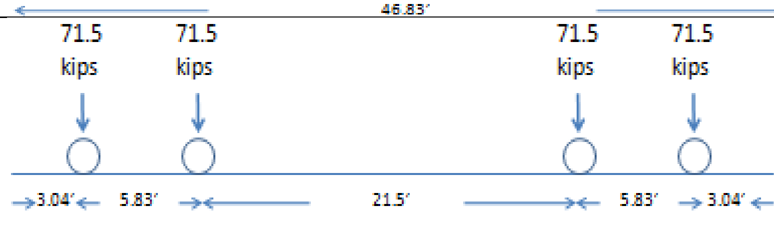
Car Type	Loading Diagram
1) 286-kips rail car used in this study	
2) Ore hopper rail car (Freight Car America)	
3) Aggregate rail car (Freight Car America)	
4) Ballast rail car (Freight Car America)	
5)Wisconsin & Southern Railroad Co. (WSOR) rail car	
6) AREMA Conference rail car 2003	

Table 4 - Configuration of rail cars tested in this project (ft)

Train type	**Rail cars configuration (ft)**
GP40-PH-20 locomotive	73 K 73 K 73 K 73 K 2nd car 9.0 28.2 9.0 16.17
GP40-PH-2B locomotive	71 K 71 K 71 K 71 K 2nd car 9.0 28.2 9.0 16.17
GP40-FH-2M locomotive	70.6 K 70.6 K 70.6 K 70.6 K 2nd car 9.0 28.25 9.0 16.17
PL42AC locomotive	72 K 72 K 72 K 72 K 2nd car 9.5 33.8 9.5 17.0
F40-PH-2 locomotive	65.4 K 65.4 K 65.4 K 65.4 K 2nd car 9.0 24.0 9.0 14.17
ALP-46(A) locomotive	51.8 K 51.8 K 51.8 K 51.8 K 2nd car 8.7 27.23 8.7 19.4
AREMA conference 286 K freight car	71.5 K 71.5 K 71.5 K 71.5 K 2nd car 5.8 21.5 5.8 6.08

RESEARCH APPROACH

Load Rating Using AREMA Specifications

AREMA was founded on October 1, 1997, by the merging of four industry-related groups: 1) the American Railway Bridge and Building Association, 2) AREA, 3) the Roadmasters and Maintenance of Way Association, and 4) the Communications and Signal Division of the Association of American Railroads. AREMA publishes the Manual for Railway Engineering with annual updates. The railway companies and consultants use the manual's recommendation as a basis for railway design and evaluation in the United States and Canada.

Loads and Forces

According to AREMA specifications, bridges shall be analyzed for different kinds of loads and their resulting forces. In this study, we consider the following load types to be consistent with the inspection reports, and each load type will be explained:

a. Dead load,
b. Live load,
c. Impact load, and
d. Wind forces.

Dead Load

The dead load represents the weight of the bridge, including the weight of the track, wood tie, ballast, deck, girder, and any other fixed loads.

Live Load

Depending on the purpose of the load rating, the live load shall be one of the Cooper E series or a specific load. In this study, the live load is the Cooper E series and 286-kips rail car. Different locomotives followed by passenger cars will also be considered live loads in this study.

Impact Load

Impact load is expressed by taking a percentage of the live load. It can be taken as the sum of vertical effect and rocking effect created by passage of train loads. Impact load resulting from vertical effects on open-deck bridges shall be determined as below (for ballasted-deck bridges, the impact load to be used shall be 90 percent of that specified for open-deck bridges). Impact load from the rocking effect (RE) shall be calculated from loads applied as a vertical force couple that each force should be taken as 20 percent of the wheel load without impact in the direction that will produce the greatest force in the member under construction.

Vertical effects, expressed as a percentage of live load applied at each rail:

a) For L less than 80 feet: $40 - \frac{3L^2}{1600}$

b) For L equal to 80 feet or more: $16 + \frac{600}{L-3}$

where L is span length (ft), center to center of supports for stringers, transverse floorbeams without stringers, longitudinal girders, and trussed (main members).

In addition, reduction factors may be applied to the vertical effects for trains at speed under 60mph. Therefore the values of the vertical effects shall be multiplied by the factor determined as follows:

$$1 - \frac{0.8}{2500}(60 - S)^2 \geq 0.2 \quad (1)$$

where S is speed in mph.

Wind Force

AREMA considered the wind force as a moving load in any horizontal direction. Wind force on the train is determined to be 200 lb per linear foot on one track applied 8 feet above the top of the rail while wind force on the bridge is determined to be 20 lb per square foot of the following surfaces (AREMA, Chapter 15, Parts 7, 7.3.3.5):[(9)]

1) For girder spans, 1.5 times the vertical projection of the span.
2) For truss spans, the vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.
3) For viaduct towers and bents, the vertical projections of all columns and tower bracing.

Rating of Existing Steel Bridges

The rating of the existing steel bridges in terms of carrying capacity shall be determined by the computation of stresses based on authentic records of the design, details, materials, workmanship, and physical condition, including data obtained by inspection (and tests if the records are not complete). If deemed advisable, field determination of stresses shall be made and the results given due consideration in the final assignment of the structure carrying capacity. For a specific service, the location and behavior under load shall be taken into account.[9] Please note that the rating of the bridge should be controlled by its weakest member.

The existing steel bridges may be assigned two types of ratings: normal and maximum. The rating or ratings assignment should be directed by the engineer. If both ratings were computed, the lesser will govern.

Normal Rating

A normal rating is defined as the load level can be carried by the expected life of the bridge. This rating can be computed with allowable reduced speed per Article 7.3.3.3, Chapter 15, for impact deduction. The speed selection shall be directed by the engineer. Allowable stresses for normal rating were specified in Section 1.4, Chapter 15, and supplemented by Article 1.3.14.3, Chapter 15. The normal rating should include the fatigue requirements of Article 7.3.4.2, Chapter 15, unless a remaining fatigue service life is computed.

The rating factor (SLN) shall be taken as the lesser of the values calculated using the following formula:

$$SLN = \frac{[S_f / 1.2] - [D + E + B + SF]}{[L + I + CF]} \tag{2}$$

$$SLN = \frac{S_f - [D + E + B + SF + 0.5W + WL + F]}{[L + I + CF + LF]} \tag{3}$$

where

SLN = service load normal rating factor,
S_f = permissible stress,
D = effect due to the dead load,
E = effect due to the earth pressure,
B = effect due to buoyancy,
SF = effect due to stream flow pressure,
L = effect due to live load,
I = effect due to impact load,
CF = effect due to centrifugal force, and
LF = effect due to longitudinal force from live load.

Please note if the rating needs to be expressed in terms of Cooper EM (E) series, the rating value shall be computed using Equation (3) with regards to Cooper EM3600 (E80) series. For other Cooper EM (E) series, the rating value is changed accordingly:

$$\text{Normal Rating} = \text{SLN} \times 360\ (\text{SLN} \times 80) \tag{4}$$

Normal ratings are evaluated with the design-allowable stresses shown in Table 5.

Maximum Rating

A maximum rating is assigned if the load level can be carried at infrequent intervals with any applicable speed restrictions. Table 6 presents the allowable stresses for maximum ratings. Fatigue need not be considered in a maximum rating.

This rating factor (SLM) shall be considered the lesser of the values calculated using the following formula:

$$SLM = \frac{S_f - [D + E + B + SF]}{[L + I + CF]} \tag{5}$$

$$SLM = \frac{1.2S_f - [D + E + B + SF + 0.5W + WL + F]}{[L + I + CF + LF]} \tag{6}$$

where *SLM* is the service load maximum rating factor.

Please note that if the rating needs to be expressed in terms of the Cooper EM (E) series, the rating value shall be computed using Equation (5) with regards to the Cooper EM3600 (E80) series. For other Cooper EM (E) series, the rating value is changed accordingly:

$$\text{Maximum Rating} = \text{SLM} \times 360\ (\text{SLM} \times 80) \tag{7}$$

This rating may be increased if the speed of traffic is reduced. A reduction of impact, as defined in Section 19.3.4, Chapter 8, AREMA Manual for Railway Engineering, can be used to recalculate the rating.[9] Maximum ratings are evaluated with the allowable stresses shown in Table 6.

Table 5 - Allowable stresses for normal rating (AREMA Manual 2010, Table 15-1-11, Page 15-1-40)[9]

Stress Area	Pounds per square inch
Axial tension, structural steel, gross section	$0.55F_y$
Axial tension, structural steel, effective net area (See Articles 1.5.8 and 1.6.5)	$0.47F_u$
Axial tension, structural steel, effective net area at cross-section of pin hole of pin connected members	$0.45F_y$
Tension in floorbeam hangers, including bending, gross section:	
Using rivets in end connections	$0.40F_y$
Using high strength bolts in end connections	$0.55F_y$
Tension in floorbeam hangers, including bending, effective net area at cross-section of pin hole of pin connected members	$0.45F_y$
Tension in floorbeam hangers, including bending, on effective net section:	$0.50F_u$
Tension in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, net section	$0.55F_y$
Tension on fasteners, including the effect of prying action:	
A325 bolts, gross section	44,000
A490 bolts, gross section	54,000
Axial compression, gross section:	
For stiffeners of beams and girders	See Article 1.7.7c
For splice material	$0.55F_y$
For compression members centrally loaded,	
when $k\ell/r \le 0.629/\sqrt{F_y/E}$	$0.55F_y$
when $0.629/\sqrt{F_y/E} < k\ell/r < 5.034/\sqrt{F_y/E}$	$0.60F_y - \left(17500\frac{F_y}{E}\right)^{3/2}\frac{k\ell}{r}$
when $k\ell/r \ge 5.034/\sqrt{F_y/E}$	$\frac{0.514\pi^2 E}{(k\ell/r)^2}$
where: $k\ell$ is the effective length of the compression member, inches, under usual conditions k = 7/8 for members with pin-end connections, k = 3/4 for members with riveted, bolted or welded end connections, and r is the applicable radius of gyration of the compression member, inches.	
Compression in extreme fibers of I-type members subjected to loading perpendicular to the web	$0.55F_y$

Table 6 - Allowable stresses for maximum rating (AREMA Manual 2010, Table 15-7-1, Page 15-7-20)[(9)]

Type	Pounds Per Square Inch
Axial tension, structural steel, gross section Axial tension, structural steel, effective net area (See Article 1.6.5) Axial tension, structural steel, effective net area at cross-section of pin hole of pin connected members	K K_1 0.82 K
Tension in floorbeam hangers, including bending, gross section: Using rivets in end connection but not to exceed Using high-strength bolts in end connection but not to exceed Tension in floorbeam hangers, including bending, effective net area at cross-section of pin hole of pin connected members: but not to exceed Tension in floorbeam hangers, including bending, on effective net section:	 0.75 K 21,600 K 28,800 0.60 K 17,300 K_1
Tension in extreme fibers of rolled shapes, girders and built-up sections, subject to bending net section	K
Tension in A325 bolts including the tension resulting from prying action produced by deformation of the connected parts, gross section	55,000
Tension in A490 bolts including the tension resulting from prying action produced by deformation of the connected parts, gross section	67,500
Axial compression, gross section: For stiffeners of beams and girders, and splice material For compression members centrally loaded, where: $kl/r \le 3388/\sqrt{F_y}$ $3388/\sqrt{F_y} < kl/r < 27111/\sqrt{F_y}$ $kl/r \ge 27111/\sqrt{F_y}$	 K K $1.091\ K - \frac{K\sqrt{F_y}}{37,300}\frac{kl}{r}$ $\frac{K}{0.55F_y}\left[\frac{147,000,000}{(kl/r)^2}\right]$
Compression in extreme fibers of I-type members subjected to loading perpendicular to web	K
Compression in extreme fibers of welded built-up or rolled beam flexural members symmetrical about the principal axis in the plane of the web (other than box type flexural members), and compression in extreme fibers of rolled channels, the larger of the values computed by the following formulas	$K - \frac{KF_y}{1.8 \times 10^9}(l/r_y)^2$ (Note 1) or $\left(\frac{K}{0.55F_y}\right)\frac{10,500,000}{ld/A_f}$ but not to exceed: K

Fatigue Rating in AREMA Specifications

According to AREMA specifications, the fatigue evaluation is required unless a bridge carries less than 5 million gross tons per year during its service life and no details of the bridge have an allowable stress range lower than Category D.

According to AREMA specifications, if there are no traffic surveys or other considerations, the number of stress cycles should be taken from Table 7. AREMA also specified the allowable fatigue stress range as shown in Table 8.

Table 7 - Number of stress cycles, N (AREMA Manual 2010, Table 15-1-7)[9]

Member Description	Span Length, L of Flexural Member or Truss or Load Condition	Constant Stress Cycles, N
Classification I		
Longitudinal flexural members and their connections. Truss chord members including end posts, and their connections	L > 100 feet	2,000,000
	L ≤ 100 feet	> 2,000,000
Classification II		
Floorbeams and their connections. Truss hangers and sub-diagonals that carry floorbeam reactions only, and their connections. Truss web members and their connections.	Two Tracks Loaded	2,000,000
	One Track Loaded	> 2,000,000
Note: This table is based on bridges designed for the live loading specified in Article 1.3.13e. For bridges designed for other live loadings see Part 9, Commentary, Article 9.1.3.13.		

Table 8 - Allowable stress range for fracture critical member (AREMA Manual 2010, Table 15-1-16)[9]

Stress Category	No. of Constant Stress Cycles, N	
	2,000,000	>2,000,000
A	24	24
B	16	16
B′	11	11
C	10	9
	12 (Note 1)	11 (Note 1)
D	8	5
E	6	2.3
E′ (Note 2)	4	1.3
F	7	6

Note 1: For transverse stiffener weld on web or flange.
Note 2: Partial length welded cover plates shall not be used on non-redundant members having flanges more than 0.8 inch thick.
Note 3: This table is based on bridges designed for the loading specified in Article 1.3.13e. For bridges designed for other live loadings see Part 9, Commentary, Article 9.1.3.13.

Load Rating Example for One of the Selected Bridges

In this study, the load rating calculations using the AREMA Specifications were performed for all selected bridges. As an example, the detailed calculations for load rating of the Main Line MP 15.95 Bridge are presented in this section. Similar rating procedure is followed for the load rating of the other four bridges. Detailed load rating results and the comparison with the FE model results can be found in the following (Please refer to "RESULTS AND COMPARISON" part).

The steel material used in the determination of the member capacity for the Main Line MP 15.95 Bridge was assumed to be fabricated from "open-hearth steel" in accordance with Inspection Reports Cycle 3.(19) The yield strength for open-hearth steel was determined to be 30 ksi from AREMA 2010 Specification 7.3.4.3.

Assumptions

The following assumptions were made during the load rating calculation:

1- Each girder carries half of the load per adjacent track.
2- Ratings for moments are at the point of maximum moment and at plate cutoff. Ratings for shear are at supports.
3- Steel members are assumed to be fabricated using "open-hearth steel".
4- Allowable stresses for ratings are as per AREMA, Article 7.3.4.3 and 1.41.
5- Overstress calculations are not included.
6- Fatigue ratings based on AREMA Specification are included.

Critical Member(s)

As shown in Figure 22, the controlling superstructure member was designated as Girder 2, Span 2, as noted in the Inspection Report Cycle 2.(25) Span length is 44.8 feet. Girder 2 is a riveted plate girder with three cover plates on top and bottom. The angles on top and bottom are riveted into the web plate, and the flanges are riveted into the angles. It has a depth of 69 in. and a width of 15 in. Figure 23 shows the cutoff points and the location of maximum moment where the methodology for load-rating checks and calculations covered in the inspection reports was applied.

Figure 22. General view of Span 2 superstructure and underside of Girder 2 of the Main Line MP 15.95 Bridge[25]

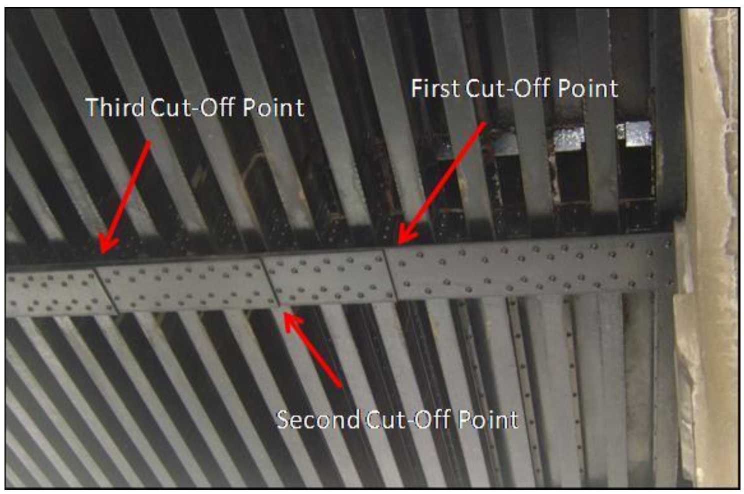


Figure 23. Cutoff points on Girder 2, Span 2, of the Main Line (ML) 15.95 Bridge[25]

The first cutoff points are located at a distance of 8.65 ft from the bearings, whereas the second and third cutoff points are located at distances of 11.0 ft and 14.4 ft from the bearings, respectively.

Since Girder 2 is in good condition, as-inspected ratings are equal to as-built ratings. Therefore, the following calculations are for both as-inspected and as-built conditions.

Load Effects

The load effects, such as moment resulting from Cooper E80 were acquired from Inspection Report First Cycle, pages 65–72.[26] The load effects due to dead load, wind load and live load of the critical members, are listed in Table 9.

Table 9 – Load effects resulting from Copper E80 Rail Car (center girder) for critical members of the Main Line MP 15.95 Bridge[26]

DEAD AND WIND LOADS RESULTING FROM E80 RAIL CAR				
	Section 1	Section 2	Section 3	Section 4
	Midspan	x=14.4'	x=11.0'	x=8.65'
Dead Load Moment (k-ft)	972.7	844.7	718.3	603.3
Wind Load Moment (k-ft)	72.0	62.5	53.4	44.9
Live Load Moment (k-ft)	3094.5	2756.3	2392.8	2009.3

Impact Factor

The impact load resulting from the sum of vertical effects and the rocking effect created by the passage of locomotives and train loads is determined by taking a percentage of the live load. It is applied vertically at the top of each rail.

The impact load resulting from vertical effects for span lengths of less than 80 ft is determined in Equation (8). Since the train speed is assumed to be 60 mph when load-rating this bridge, the full impact factor without reduction is used.

$$Vertical\ effects = 40 - \frac{3L^2}{1600} \tag{8}$$

Equation (9) shows the formula for calculating the impact factor in accordance with AREMA 2010, Article 1.3.5:

$$I = \left[RE + 40 - \frac{3L^2}{1600} \right] \tag{9}$$

where $RE = \pm 20\%$

The impact load resulting from the rocking effect, RE, is created by the transfer of load from the wheels on one side of the rail car to the other side from periodic lateral rocking of the equipment. RE is calculated as a vertical force couple, each being 20 percent of the wheel load without impact, acting downward on one rail and upward on the other. The couple is oriented in a way that creates the greatest force in the member. On the other hand, the impact factor needs to be modified due to the ballast deck effect with a factor of 0.9 (AREMA Chapter 15, Article 1.3.5 (b)).[9] Taking an example from Inspection Report (Inspection Report Cycle 2, Polytran Engineering Associates, PC, 2001, pages 2–15).[25], the impact factor on the track can be calculated as follows:

$$I = 0.9 \cdot \left[\pm 20 + 40 - \frac{3 \cdot (13)^2}{1600} \right] = 53.71\% \quad and \quad 17.71\%$$

To obtain the impact factor of the floorbeam, the exact location of the track must be determined. Since the eccentricity of the tracks is 1.5 in. to the north over the floor beams. It is calculated that one track is located at 3.87 ft from the left support R_1, the other is located at 4.13 ft from right support R_2, as shown in Figure 24.

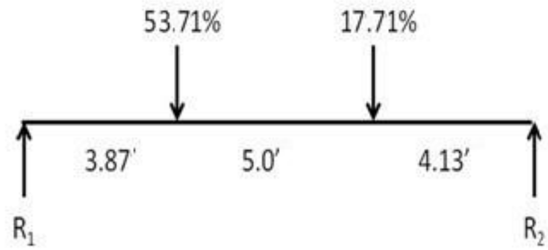


Figure 24. Impact load configuration on the floor beam

Therefore, based on the loading configuration shown in Figure 24, the impact factor of floor beam can be calculated as follows:

$$I = \frac{\left[53.71 \times (5 + 4.13) + 17.71 \times 4.13\right]}{13} = 43.34\%$$

Cooper E80 Load Rating

After obtaining the load effect and impact factor of the member that needs to be load rated, we can perform the load rating on the critical member. Load ratings resulting from Cooper E80 Rail Car represented in Table 10 were also acquired from Inspection Report First Cycle, pages 65–72, whereas the section properties and dimensions were taken from Inspection Report Cycle 2, pages 2–23.[25,26] The live load moment due to 286-kips rail car loading were determined using an Excel program, “QuickBridge,” developed by Professor Noyan Turkkan of the Ecole de Genie (School of Engineering), Universite de Moncton, Canada.

Table 10 - Cooper E80 load rating results (center girder)

	Section 1	Section 2	Section 3	Section 4
	Midspan	*x=14.4'*	*x=11.0'*	*x=8.65'*
$M_{LL\text{-}E80}$ (k-ft)*	3094.5	2756.3	2392.8	2009.3
$M_{LL\text{-}286}$ (k-ft)**	2350.0	2079.3	1753.5	1488.5
Normal Moment Rating	E 64	E 62	E 58	E 52
Maximum Moment Rating	E 102	E 98	E 92	E 84
Section Modulus in^3	3420.9	2940.5	2408.6	1878.5
$F_{y\ NORMAL}$ (ksi)	16.5	16.5	16.5	16.5
$F_{y\ MAXIMUM}$ (ksi)	24.0	24.0	24.0	24.0
$M_{Capacity\text{-}NORMAL}$ (k-ft)	4703.7	4043.2	3311.8	2582.9
$M_{Capacity\text{-}MAXIMUM}$ (k-ft)	6841.8	5881.0	4817.2	3757.0
$V_{Allowable\text{-}NORMAL}$ (ksi)	10.5	Only one section is considered for Shear Rating		
$V_{Allowable\text{-}MAXIMUM}$ (ksi)	18.0			
$V_{Capacity\text{-}NORMAL}$ (ksi)	543.4	Shear Area is 51.8 in^2		
$V_{Capacity\text{-}MAXIMUM}$ (ksi)	931.5			
Shear Normal Rating	E 77			
Shear Maximum Rating	E 144			

* From Table 9

**Calculated using "QuickBridge"

Equivalent Cooper-E load resulting from 286kipsrail car

The equivalent Cooper E load is a measure of live load effects on a bridge member resulting from rail car other than Cooper E. In this example, the equivalent Cooper E load is calculated for various sections. The maximum moment used in the following calculations were obtained from Table 10.

Section 1 (Maximum Moment Point)

First, the ratio between the maximum moments of a 286-kips rail car and Cooper E80 is computed as follows:

$$Moment\ Ratio = \frac{M_{LL-286kips}}{M_{LL-E80}} = \frac{2350}{3094.5} = 0.75$$

$$286\ kips\ equivalent\ Copper\ E\ load = moment\ ratio \times 80 = 0.756 \times 80 = 60.4$$

Then, the moment ratio between the 286-kips rail car and Cooper E80 is multiplied by the heaviest axle load of the Cooper E80 rail car, which is 80 kips, and the result is 60.48. Therefore, the equivalent Cooper E load from 286 kips rail car is E60.

Section 2 ($x = 14.4'$)

$$Moment\ Ratio = \frac{M_{LL-286kip}}{M_{LL-E80}} = \frac{2079.3}{2756.3} = 0.75$$

$$286\ kips\ equivalent\ Copper\ E\ load = moment\ ratio \times 80 = 0.75 \times 80 = 60.3$$

Similarly, the equivalent Cooper E load is E60.

Section 3 ($x = 11.0'$)

$$Moment\ Ratio = \frac{M_{LL-286kips}}{M_{LL-E80}} = \frac{1753.5}{2392.8} = 0.73$$

$$286\ kips\ equivalent\ Copper\ E\ load = moment\ ratio \times 80 = 0.736 \times 80 = 58.9$$

Similar to the calculations shown in previous sections, the equivalent Cooper E load is E59.

Section 4 ($x = 8.65'$)

$$Moment\ Ratio = \frac{M_{LL-286kips}}{M_{LL-E80}} = \frac{1488.5}{2009.3} = 0.74$$

$$286\ kips\ equivalent\ Cooper\ E\ Load = moment\ ratio \times 80 = 0.74 \times 80 = 59.2$$

Therefore, the equivalent Cooper E load is E59.

Table 11 presented a summary of 286 kips equivalent Cooper E load. It is common to have different equivalent load ratings for the same girder because of the differences in the axle spacing and positioning of the rail cars at the different points of output generation. This suggests that the load ratings can be different depending on the location of the cutoff point along the girder. It is also possible that one axle of the 286-kips car "falls off" the bridge on the approach and would not be included in loading the bridge.

Table 11 - 286 kips equivalent Cooper E load calculation

Simple beam analysis	Midspan	14.4' from support	11' from support	8.65' from support
286 kips equivalent Cooper E load	E-60	E-60	E-59	E-59

Sensor Instrumentation and Field Testing

Field tests are included as part of the evaluation process to confirm results from analytical models and AREMA rating procedures. Field-testing was performed for all five selected bridges. The target bridges were tested to obtain various structural responses such as strain, deflection, and velocity. The testing results will be used to evaluate the performance of the bridge and improve the accuracy of the analysis model.

Field-Testing Objectives

The purpose of the field-testing can be summarized as follows:

1. Obtain structural response (strains, deflections and velocity) under static and dynamic rail car loading,
2. Analyze the testing data to evaluate the overall condition of the bridge,
3. Evaluate the impact factors under various speeds, and
4. Validate and calibrate the FE model.

Testing Equipment

Structural Testing System

The Structural Testing System (STS) is a modular data acquisition system manufactured by Bridge Diagnostics, Inc. (BDI), of Boulder, Colorado. The system consists of a main processing unit that samples data, junction boxes, and strain transducers. The strain transducers are mounted to structural elements with C-clamps or bolted to epoxied tabs. Each transducer has a unique identification number and a microchip to help identify it easily in the system. The transducer calibration factors are stored in the configuration files and are applied automatically.

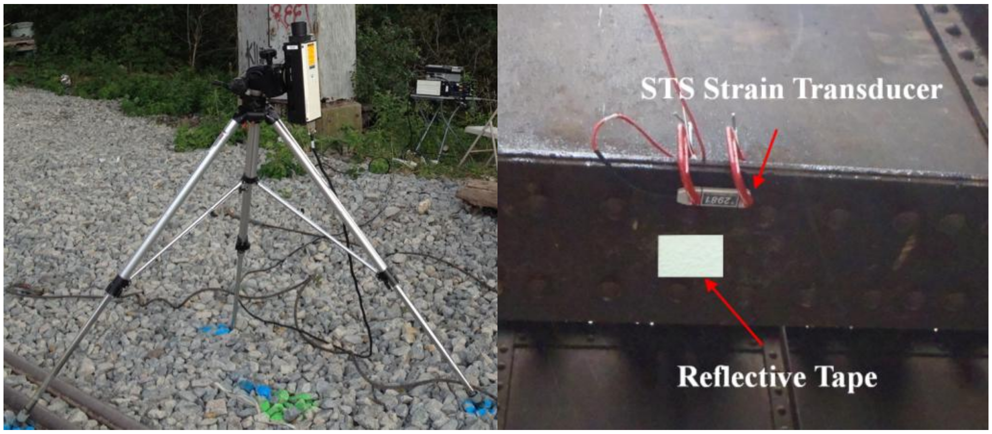
The STS consists of strain transducers, junction node, and the main STS unit as shown in Figure 25. Each test is assigned to an automatic file number, and the test is initiated using a trigger button called the clicker. Once the test is completed, the data can be downloaded from the STS unit to a laptop computer.

(a) *(b)* *(c)*

Figure 25.*(a)*STS strain transducer;*(b)*junction node, and *(c)*main unit

Laser Doppler Vibrometer

The Laser Doppler Vibrometer (LDV), shown in Figure 26, is a noncontact measuring device that measures displacement and the velocity of a remote point. A change in the distance between the laser head and the reflective target will produce a Doppler shift in the light frequency that is decoded into displacement and velocity. The system is composed of three parts: 1) the helium neon Class II laser head, 2) the decoder unit, and 3) the reflective target attached to the structure. The laser head is mounted to a tripod that is positioned underneath the target. The reflective target, typically retro-reflective tape, provides the strongest signal. The signal strength is read on a scale on the laser head. The tripod is adjusted to maximize the signal prior to a test run.



(a) *(b)*

Figure 26. *(a)* Laser Doppler Vibrometer and *(b)* locations of reflective targets for measuring deflections

<u>Field Testing of the Main Line MP 15.95 Bridge</u>

The sensor instrumentation at this structure is focused on the center girder on Span No. 2, since this member has the lowest rating based on the Inspection Report Cycle 3.(19)

The behavior of the center girder will be evaluated at the cutoff locations and at midspan. For the exterior girders, strain gage installation was not possible, since the girders were encased in concrete and the girder flanges are not accessible. Figure 27 shows the testing set-up and preparation during the installation of the sensors. Table 12 and Figure 28 show the location of the 12 strain transducers and five reflective tapes that were instrumented on the Main Line MP 15.95 Bridge.

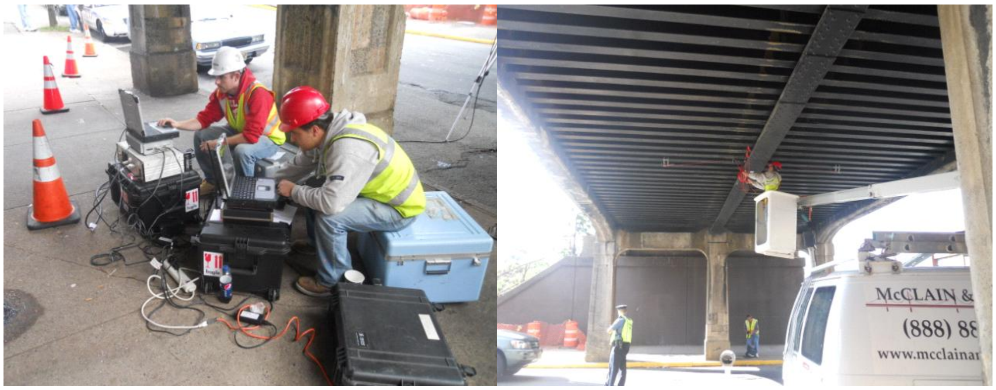


Figure 27. Sensor instrumentation and test equipment at the *Main Line MP 15.95 Bridge*

Table 12 - Sensor ID numbers and locations at the *Main Line MP 15.95 Bridge*

Sensor No.	Sensor ID Number	Sensor Location
1	2047	Floor beam (first cutoff point, G2-G3)
2	2049	Girder 2, first cutoff point, first plate
3	2050	Girder 2, first cutoff point, second plate
4	2042	Floor beam (first cutoff point, G1-G2)
5	2045	Girder 2, second cutoff point, second plate
6	2046	Girder 2, second cutoff point, third plate
7	2491	Girder 2, third cutoff point, third plate
8	2490	Girder 2, third Cutoff point, fourth plate
9	2493	Girder 2, between midspan and third cutoff
10	2487	Girder 2, midspan
11	2488	Floor beam (midspan, G2-G3)
12	2484	Floor beam (midspan, G2-G1)

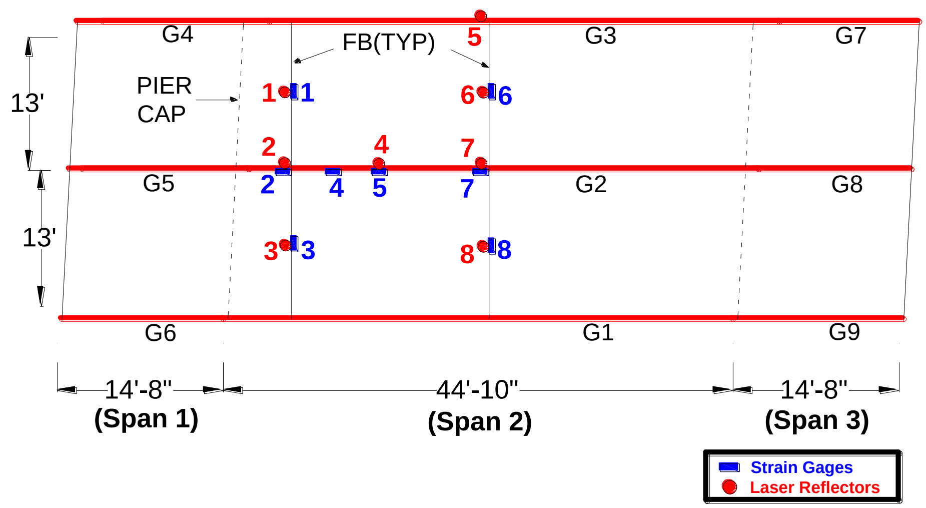


Figure 28. Sensor locations on the plan view for the *Main Line MP 15.95 Bridge*

After the installation of sensors, the tests were conducted with the scheduled passenger trains from Paterson to Hawthorne as shown in Table 13. During the first run, only Laser Doppler Unit measurements could be collected.

Table 13 - Information about the tested trains at the *Main Line MP 15.95 Bridge*

Train Number	Run Number	Direction	Time	Type	Speed (mph)
76	1	Hawthorne-Paterson	12:39 p.m.	GP40-PH-2B	31.4
1717	2	Paterson-Hawthorne	12:58 p.m.	PL-42AC	30.9
1716	3	Hawthorne-Paterson	1:39 p.m.	GP40PH-20	37.4
1719	4	Paterson-Hawthorne	1:58 p.m.	GP40FH-2M	31.4

The strain and deflection data measurements that recorded for the runs with the train going in the same direction show similar values. Figure 29 shows typical strain data obtained from the STS strain transducers. Figure 30 illustrates the typical deflection measurement results. Please refer to Appendix A for more detailed experimental data from various bridge tests.

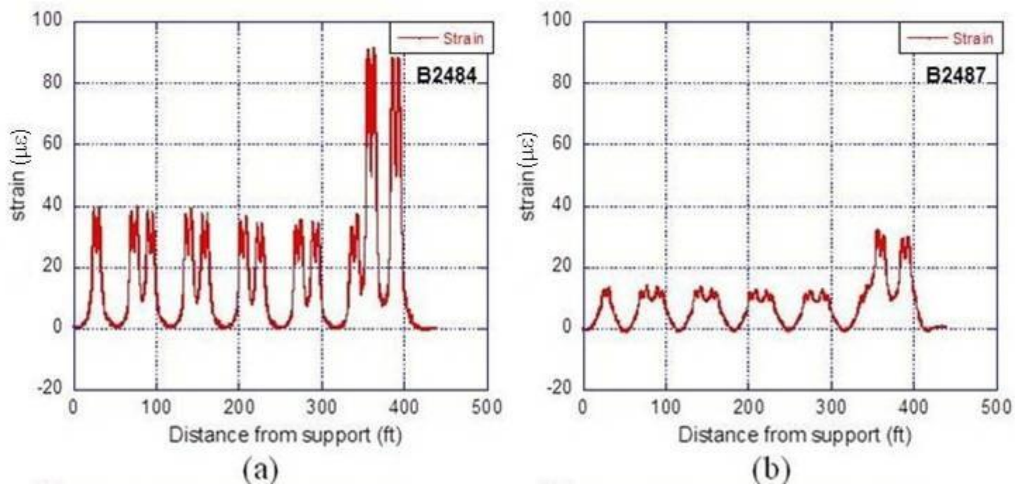


Figure 29. Typical strain data at strain transducer location *(a)* B2484, *(b)* B2487, *(c)* B2488, and *(d)* B2491 for the *Main Line MP 15.95 Bridge*

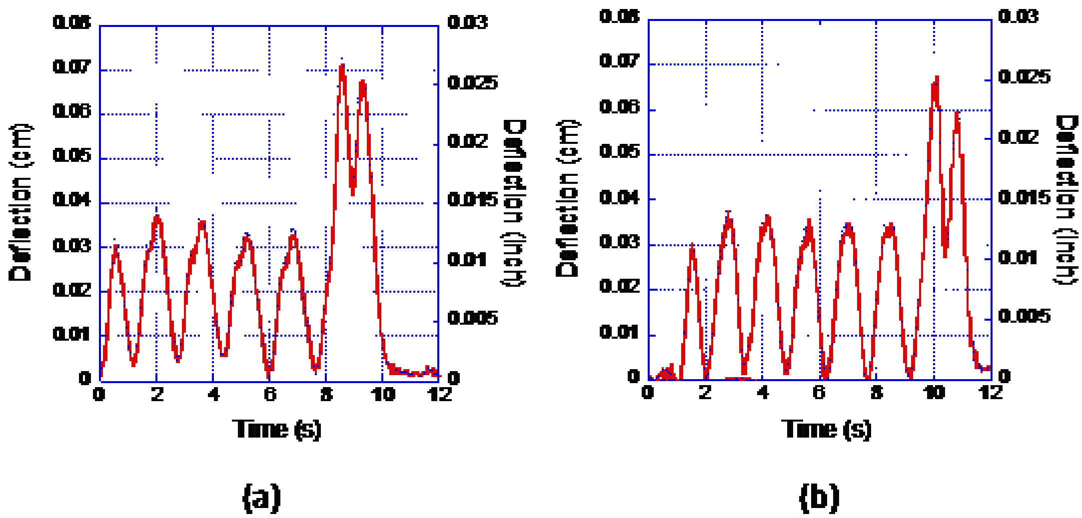


Figure 30. Deflection measurements: *(a)* Test Run #1 (Reflective Tape 5), *(b)* Test Run#2 (Reflective Tape 4) for the *Main Line MP 15.95 Bridge*

<u>Field Testing of the Main Line MP 15.14 Bridge</u>

As shown in Figure 18, this bridge contains seven spans, where two of these spans pass over Straight Street (Spans 2 and 3), while two other spans pass over 21st Street. The sensor instrumentation was focused on Span 1, which is over the sidewalk at the east abutment, and on Span 2, which is over the Straight Street. Figure 31 shows a view of Spans 1, 2, and 3, looking north.

Figure 31. Instrumented Spans 1, 2, and 3 of the Main line MP 15.14 Bridge.

A total of 20 STS strain transducers and 13 reflectors were installed on this bridge. Figure 32 shows the locations of the strain transducers and reflective-tape targets that were used on the members of the ML 15.14 bridge structure. Table 14 lists the floor beams that were selected for the installation and their respective section geometry.

Table 14 - Instrumented floor beam of the Main line MP 15.14 Bridge (refer to the original bridge drawings for the floor beam notation)

Span Number	**Floor Beam Number**	**Section Dimensions (I-Beam)**
1	F226	15"×50"×15'-5 7/16"
1	F227	15"×50"×15'-4 15/16"
1	F228	15"×50"×15'-5 3/4"
1	F229	15"×50"×13'-11 5/6"
1	F349	15"×50"×13'-11 15/16"
1	F472	15"×50"×14'-5 7/16"
2	F574	15"×50"×14'-11 13/16"
2	F180	15"×50"×14'-6 1/16"
2	F264	15"×50"×14'-3 1/8"
2	F404	15"×50"×14'-8 13/16"
2	F534	15"×50"×14'-5 3/16"

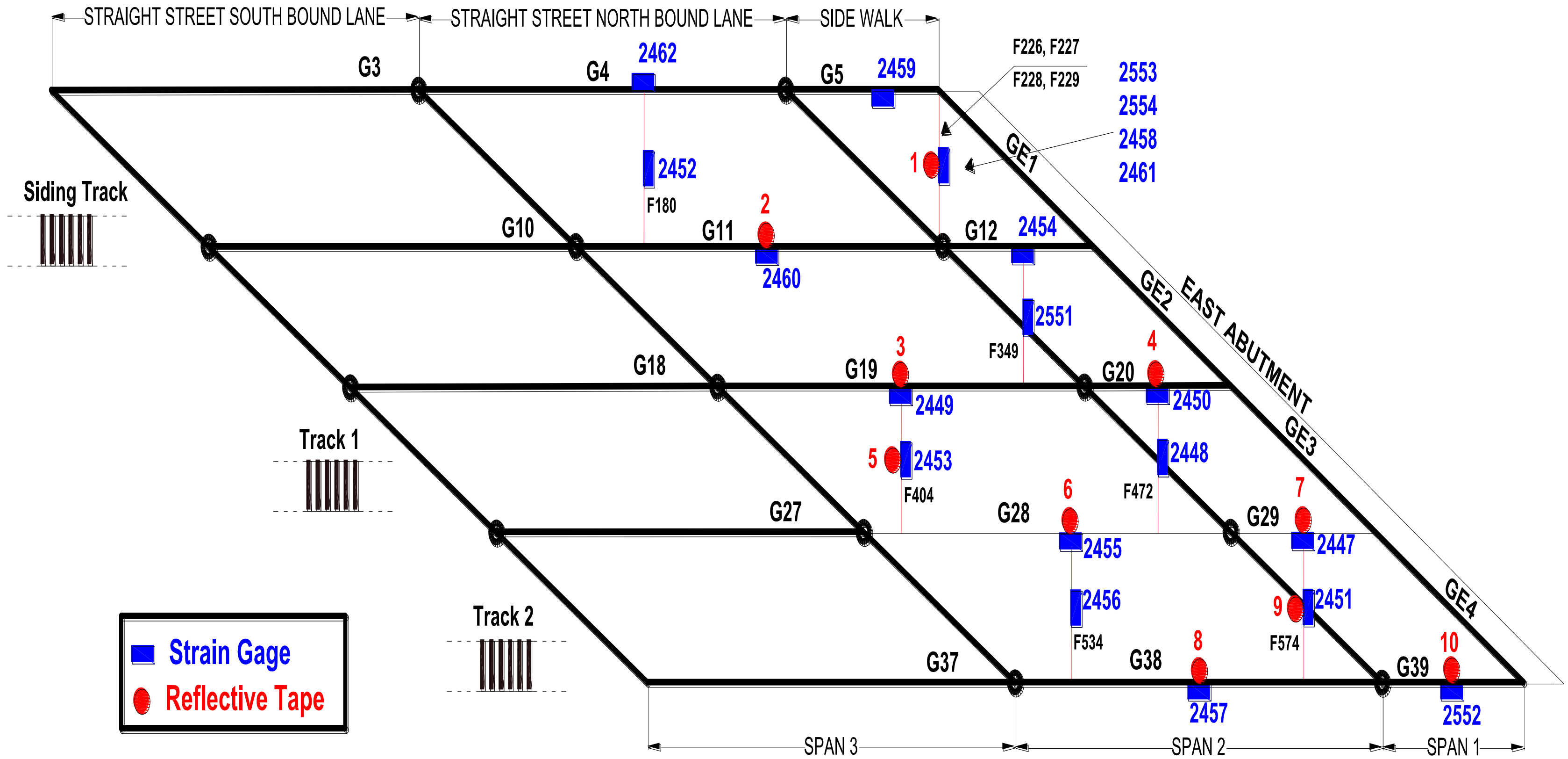


Figure 32. Sensor locations on the plan view of the Main Line 15.14 Bridge (not to scale)

Table 15 lists the information on the test train. There were total 3 runs that were recorded during the field-testing. Both directions were included in the 3 runs. Figure 33 shows typical strain data obtained during the field tests. Please refer to Appendix B for more detailed information of the experimental data.

Table 15 - Performed tests with the regular train traffic at the Main line MP 15.14 Bridge

Run Number	Information About the Tested Train				Tests	
	Direction	**Time**	**Train Type**	**Speed (mph)**	**STS Strain Transducer**	**Laser Doppler Unit**
1	Clifton-Paterson	11:51 a.m.	PL42-AC	31.1	All sensors	N/A
2	Paterson-Clifton	12:03 p.m.	F40PH-2CAT	32.3	All sensors	Reflective Target#10
3	Clifton-Paterson	01:17 p.m.	GP40FH-2M	35.9	All sensors	Reflective Target#8

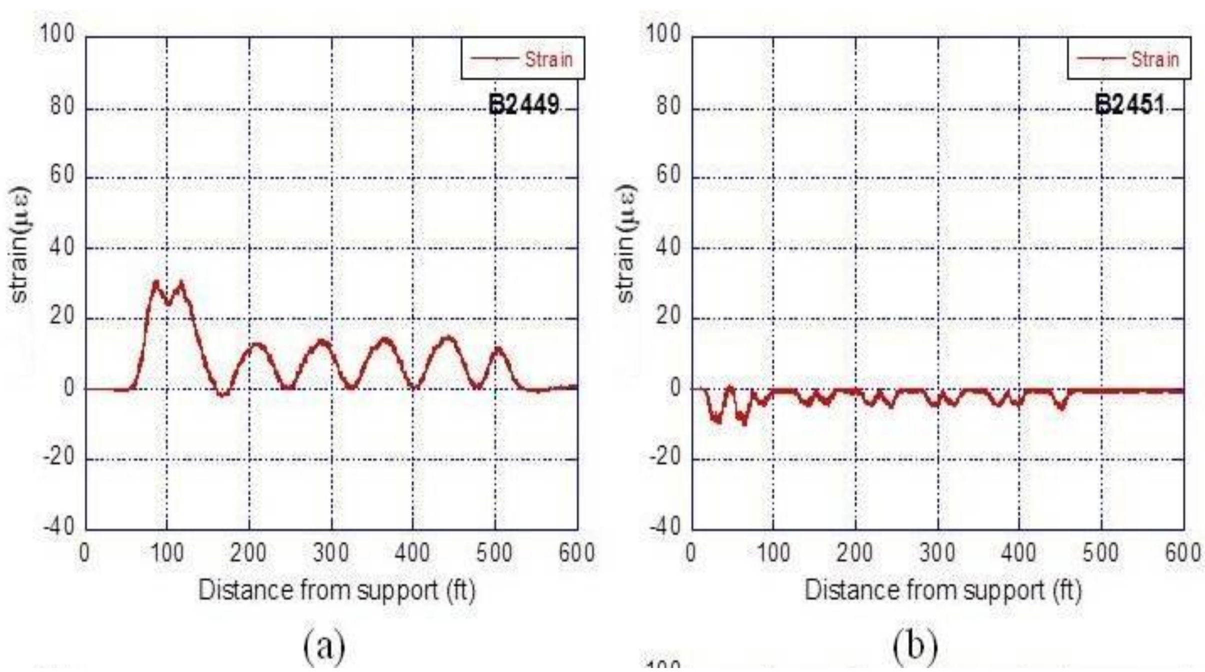


Figure 33. Typical strain data collected for Main Line MP 15.14

Field Testing of the Bergen County Line MP 5.48 (HX Draw) Bridge

The sensor instrumentation at this structure was mainly focused on the north girder in Span No. 3 (approach span) below Track 2. This girder has the lowest as-inspected rating, which are E27 at normal level and E43 at the maximum level. Strain transducers were also installed on the south girder under Track 2, the girders under Track 1, and the girders in Span No. 2, to gain a thorough understanding of the structural response and load distribution of the bridge. Moreover, based on the preliminary calculation performed by NJ Transit, the end floor beam of Span No. 9 (approach span) and stringers in Span 12 (tower span) were also selected for testing. Figure 34 through Figure 38 show the locations of sensors on the desired spans specified by the sensor number.

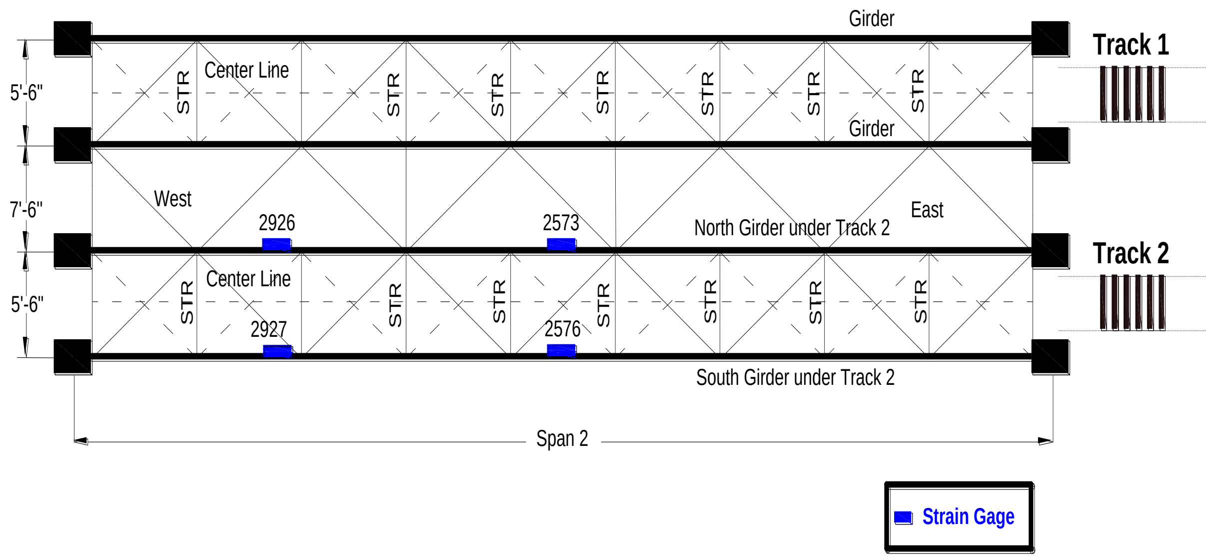


Figure 34. Layout of the strain transducers installed in Span 2 *of the Bergen County Line MP 5.48 (HX Draw) Bridge*

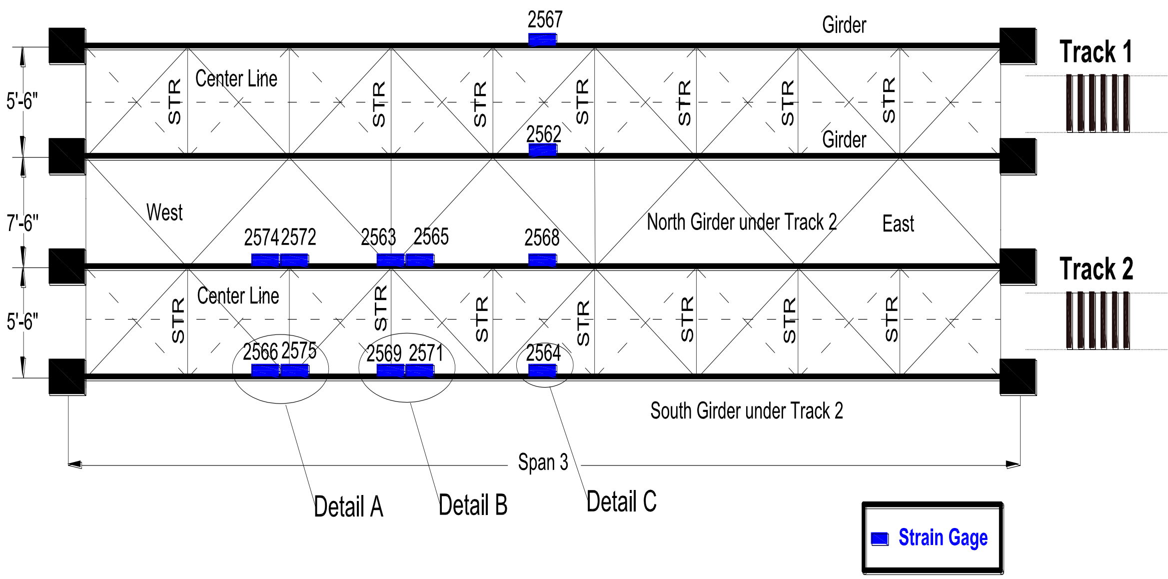


Figure 35. Layout of the strain transducers installed in Span 3 of *the Bergen County Line MP 5.48 (HX Draw) Bridge*

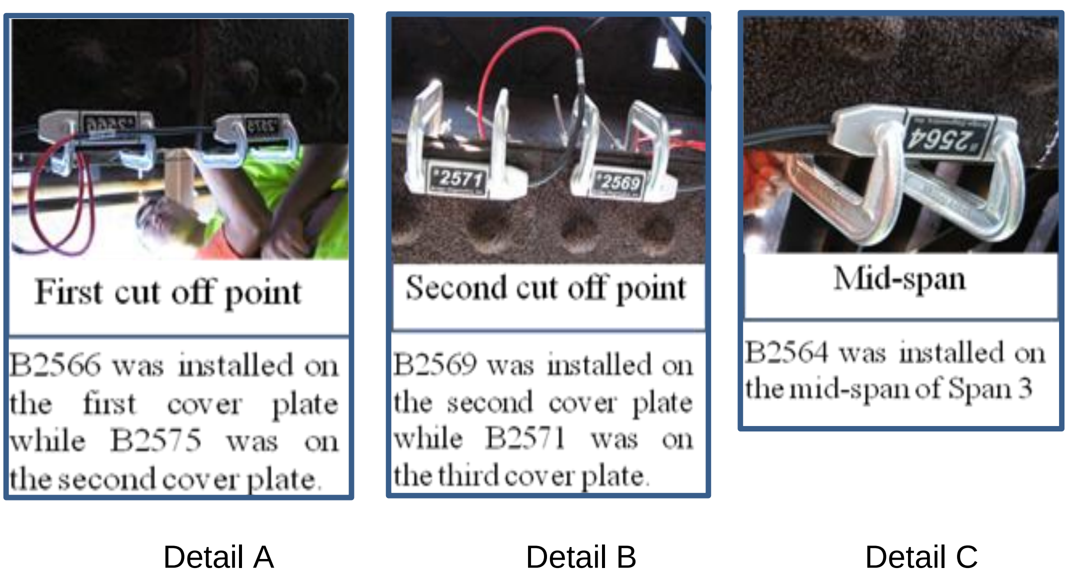


Figure 36. Details information in Span 3 of *Bergen County Line MP 5.48 (HX Draw) Bridge*

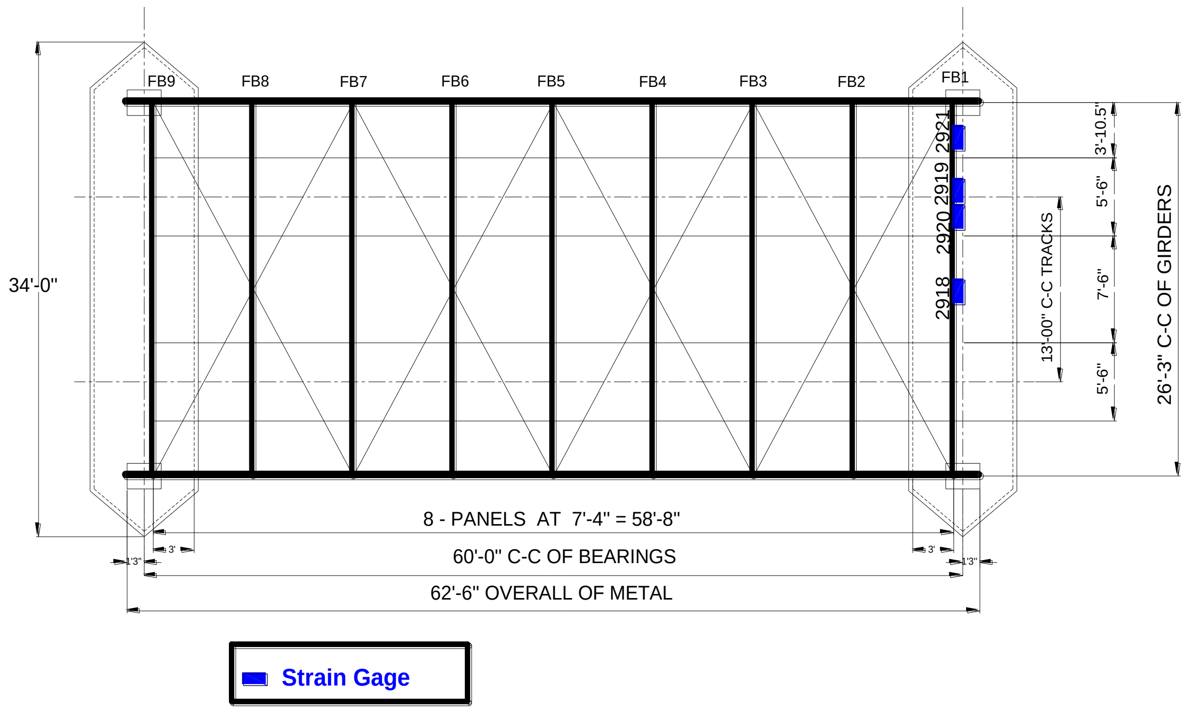


Figure 37. Layout of the strain transducers installed in Span 9

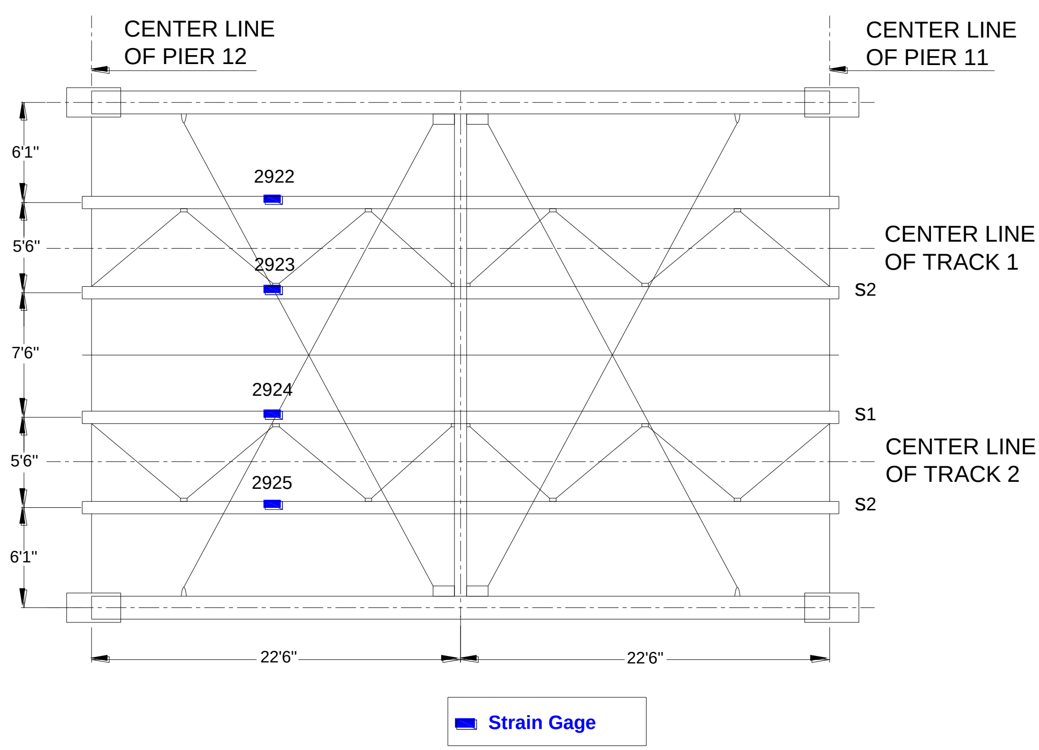


Figure 38. Layout of the strain transducers installed in Span 12 *of the Bergen County Line MP 5.48 (HX Draw) Bridge*

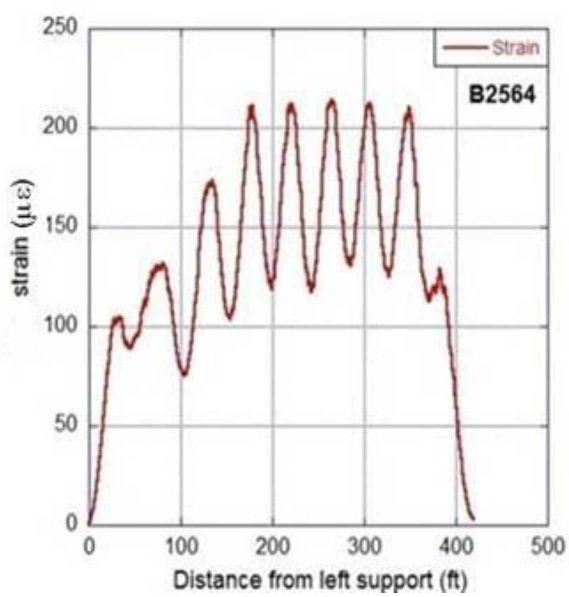
Table 16 shows the information for each run of the test, and Table 17 shows the configurations of the six test trains. Moreover, Figure 39 shows typical strain data collected when the train passed over the bridge at a speed of 10 mph. The maximum strain collected from the field is equal to 220 με, which is equivalent to a stress of 6.4 ksi. Please refer to Appendix C for more detailed information on additional experimental data.

Table 16 - Tested information for *of the Bergen County Line MP 5.48 (HX Draw) Bridge*

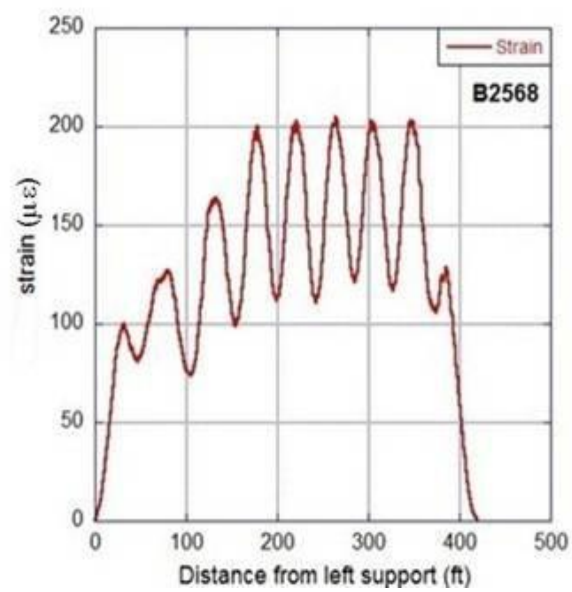
Run#	Time (a.m.)	Track	Direction	Type	Note
1	8:32	1	Westbound	Freight	
2	8:45	2		Freight	Static test
3	8:51	2		Freight	Static test
4	8:58	2		Freight	Static test
5	9:03	2	Westbound	Freight	10mph
6	9:10	2		Freight	Static test
8	9:18	2		Freight	Static test
10	9:25	2		Freight	Static test
11	9:30	2	Westbound	Freight	10mph
12	9:36	2	Eastbound	Freight	20mph
13	9:36	2	Westbound	Freight	20mph
14	9:40	2	Eastbound	Freight	25mph
15	9:45	2	Westbound	Freight	25mph
16	9:48	2	Eastbound	Freight	10mph
17	9:51	2	Westbound	Freight	10mph
18	9:54	2	Eastbound	Freight	20mph
19	9:58	2	Westbound	Freight	20mph
20	10:00	2	Eastbound	Freight	25mph
21	10:04	2	Westbound	Freight	25mph
23	10:21	1		Freight	Static test
24	10:28	1		Freight	Static test
25	10:29	1		Freight	Static test
26	10:30	1		Freight	No train passed
27	10:31	1		Freight	Static test
29	10:49	1	Westbound	Freight	10mph
30	10:53	1	Eastbound	Freight	10mph
31	10:56	1	Westbound	Freight	20mph
32	11:00	1	Eastbound	Freight	20mph
33	11:06	1	Westbound	Freight	25mph
34	11:09	1	Eastbound	Freight	25mph

Table 17 - Tested 286-kips railcars configuration *for the Bergen County Line MP 5.48 (HX Draw) Bridge*

Train		Weight Ticket (lbs)	Car Weight (lbs)	Net (lbs)	Outside Length	Axle Spacing	Length between Front and Rear Axle Sets
NS	0000994145	213,600	68,900	283,400	45'-11"	70"	32'-4"
NS	0000994257	213,600	68,900	282,500	45'-11"	70"	32'-4"
NS	0000994285	213,600	68,600	282,200	45'-11"	70"	32'-4"
NS	0000994671	213,600	57,600	271,200	45'-1"	70"	31'-6"
NS	0000994782	213,600	58,900	272,500	45'-1"	70"	31'-6"
NS	0000994198	213,600	69,100	282,700	45'-11"	70"	32'-4"



(a)



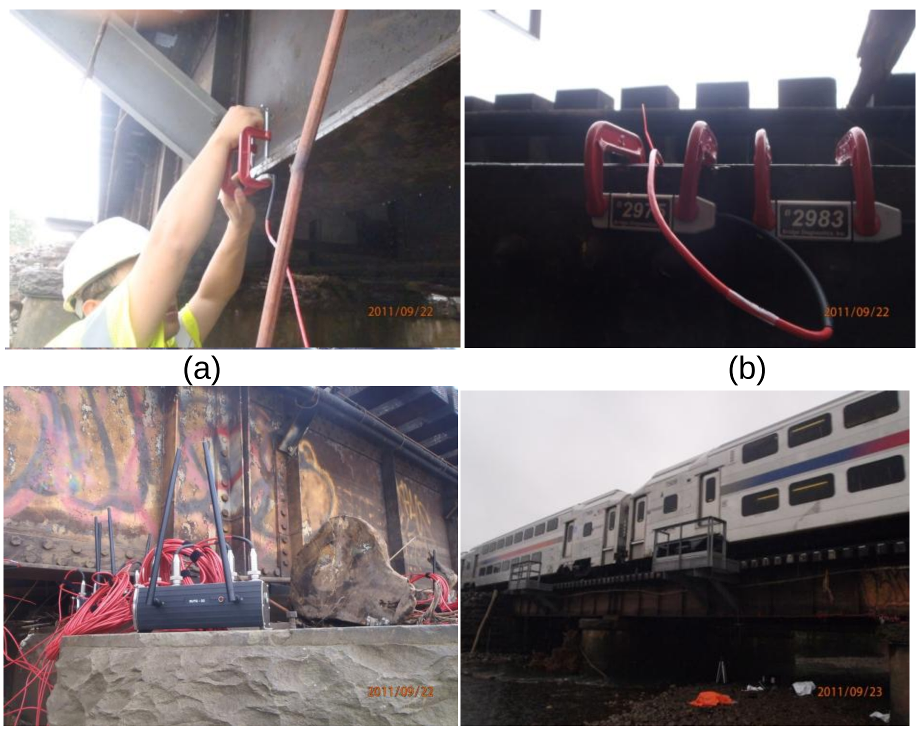
(b)

Figure 39. Strain data collected from Test Run #5 for a train speed of 10-mph at transducer locations: *(a)* B2564 and *(b)* B2568

Field Testing of the Raritan Valley Line MP 31.15 (Middle Brook) Bridge

Since Girder G8 in Span 2, right beneath the active track, was rated as the critical member in the Inspection Report Cycle 4, Chas. H. Sells, Inc., 2007, (E52 at normal level and E75 at the maximum level; E62 at first cutoff point and E55 at second cutoff point), the field instrumentation of sensors was focused on this girder. Strain transducers were also installed on Girders G5–G7 in Span 2 and Girders G5–G8 in Span 3 to provide an additional understanding of the structural response of the bridge. Figure 40 shows photos taken at various stages of the field-testing set-up including the installation of sensors and when tested train was passing over the bridge.

Figure 41 and Figure 42 show a layout of the bridge and locations of the strain transducers that were installed on Span 2 and Span 3 of the Raritan Valley Line MP 31.15 Bridge, respectively. It is noted that the reference Section A-A shown in Figure 41 is taken at the midspan section (Girders G7 and G8), while reference section B-B is taken at the first cutoff point (about 5 ft from the support end of the girder), and reference section C-C is taken at the second cutoff point (about 8 ft and 8.5 in. from the support of the girder). For Span 3, sensors are installed at the midspan of each girder under the active track (G5 through G8). Table 18 and Table 19 show a summary of the tested train information for the tests performed on 9/30/2011 and 9/23/2011, respectively.



(a) (b) (c) (d)

Figure 40. Sensor installation *of the Raritan Valley Line MP 31.15 (Middle Brook) Bridge: (a)* Sensor installation, *(b)* Installed sensors on the bottom of girder, *(c)* Junction nodes on the pier, *(d)* Tested train on the bridge

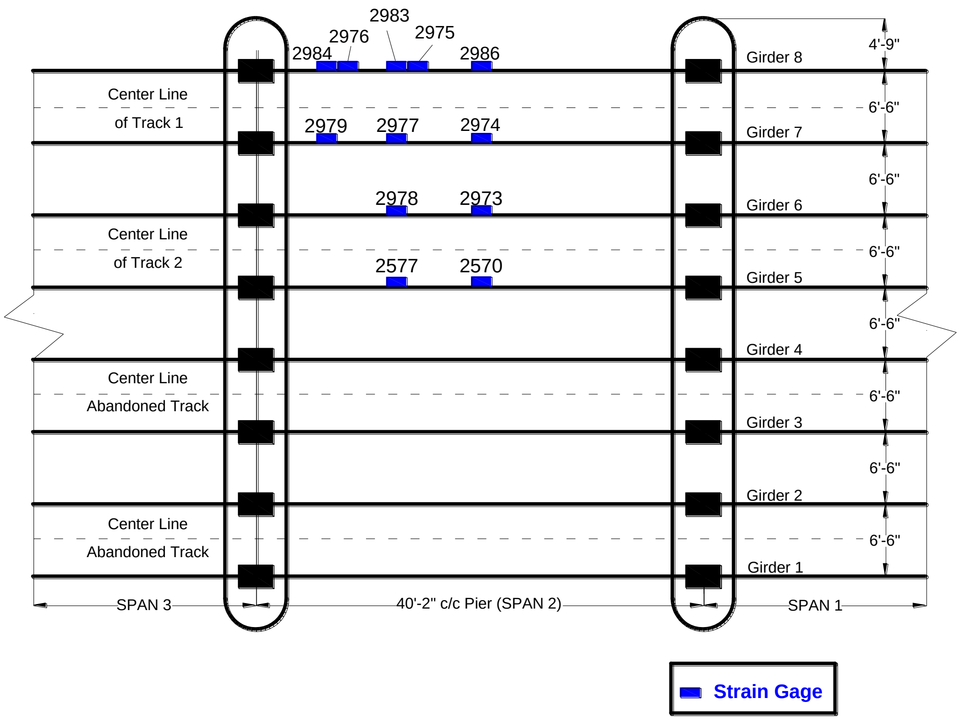


Figure 41. Location of strain transducers in Span 2 *of the Raritan Valley Line MP 31.15 (Middle Brook) Bridge*

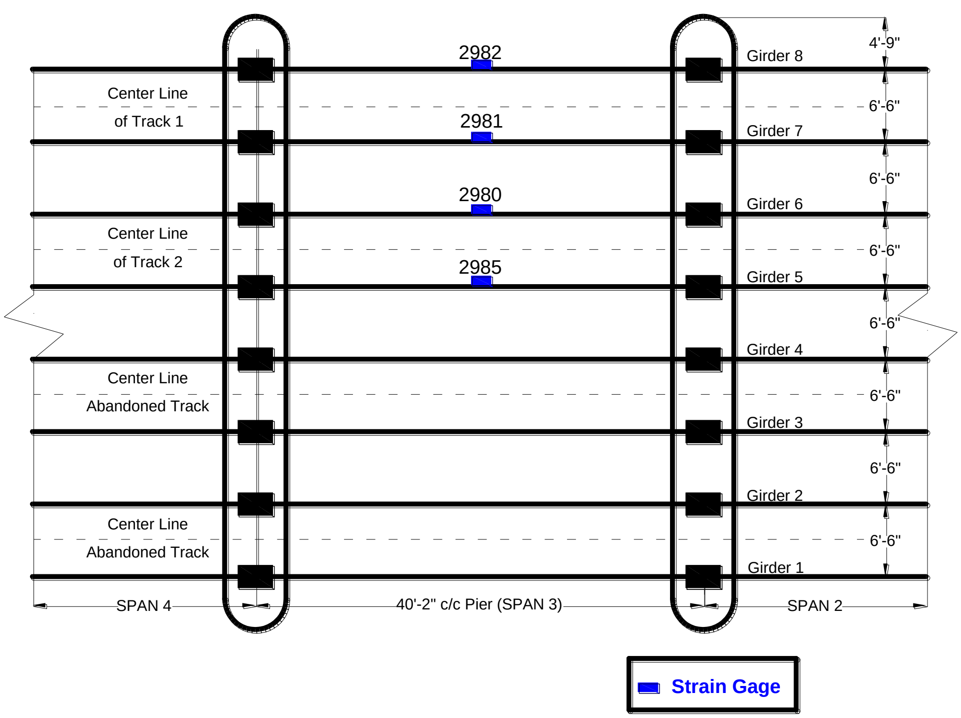


Figure 42. Location of strain transducers in Span 3 *of the Raritan Valley Line MP 31.15 (Middle Brook) Bridge*

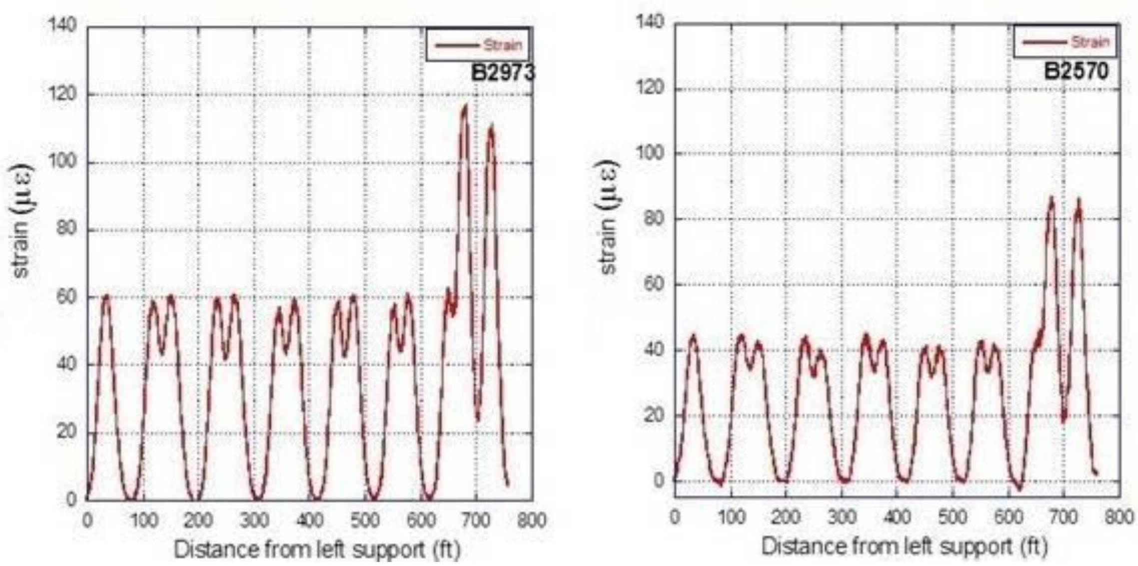
Table 18 - Train information for the Raritan Valley Line on test dated 09/30/2011

Run #	Arrival Time	Direction	Track	Train #	Laser	File Name
7	8:44 a.m.	Bound Brook-Bridgewater (W)	1	5413	N/A	0923-2
8	8:52 a.m.	Bridgewater-Bound Brook (E)	2	5426	N/A	0923-3
9	9:49 a.m.	Bound Brook-Bridgewater (W)	1	5415	G8	0923-4
10	9:56 a.m.	Bridgewater-Bound Brook (E)	2	5730	G8	0923-5
11	10:51 a.m.	Bound Brook-Bridgewater (W)	1	5719	G8	0923-6
12	10:56 a.m.	Bridgewater-Bound Brook (E)	2	5432	G6	0923-7
13	11:49 a.m.	Bound Brook-Bridgewater (W)	1	5421	G8	0923-8
14	11:56 a.m.	Bridgewater-Bound Brook (E)	2	5434	G6	0923-9
15	12:49 p.m.	Bound Brook-Bridgewater (W)	1	5423	G7	0923-10
16	12:55 p.m.	Bridgewater-Bound Brook (E)	2	5736	G5	0923-11
17	1:48 p.m.	Bound Brook-Bridgewater (W)	1	5725	G7	0923-12
18	1:56 p.m.	Bridgewater-Bound Brook (E)	2	5438	G5	0923-13
19	2:51 p.m.	Bound Brook-Bridgewater (W)	1	5427	G8	0923-14
20	2:56 p.m.	Bridgewater-Bound Brook (E)	2	5440	G5	0923-15

Table 19 - Tested train information for the Raritan Valley Line MP 31.15 (Middle Brook) Bridge on 09/23/2011

	Information About the Tested Train			
	Direction	**Time**	**Speed (mph)**	**Track #**
Run #1	Bound Brook – Bridgewater	9/22/2011 15:55	34.2	2
Run #2	Bridgewater – Bound Brook	9/22/2011 16:00	N/A	1
Run #3	Bound Brook – Bridgewater	9/22/2011 16:25	N/A	2
Run #4	Bridgewater – Bound Brook	9/23/2011 10:10	36	1
Run #5	Bound Brook – Bridgewater	9/23/2011 11:03	35.5	1
Run #6	Bridgewater – Bound Brook	9/23/2011 11:29	37	1

Figure 43 shows typical strain data collected from tests performed on the Raritan Valley Line MP 31.15 (Middle Brook) Bridge. Please refer to Appendix D for more detailed information on additional experimental data for this bridge.



(a) (b)

Figure 43. Strain data measured from Test Run #1 at Strain Transducer No. *(a)* B2973, *(b)* B2570

Field Testing on the North Jersey Coast Line MP 0.39 (River Draw) Bridge

The North Jersey Coast Line MP 0.39 (River Draw) Bridge is a steel-truss swing span flanked by 28 simply supported steel-deck girder spans. The bridge was erected in 1906 and carries two electrified tracks over the Raritan River between Perth Amboy and South Amboy, New Jersey.

The sensor instrumentation at this structure focused mainly on Girder 5 through Girder 8 in Span 26 (one of typical 88 ft approach span), since these members rated lowest as inspected (E47 at normal level and E70 at maximum level). Figure 44 shows the elevation and bottom views of a typical approach span (span length is 88 ft). Sensors were also instrumented on the end floorbeam of Span 20. Figure 45 and Figure 46 show the locations of the strain transducers installed on Span 26 and Span 20, respectively. Please note that sensor 3236 is located at the midspan of the end floorbeam, while sensors 3228, 3217, and 3229 are located at the cutoff point.

(a) (b)

Figure 44. Elevation view (a) and bottom view (b) of a typical approach span (span length = 88 ft) of the North Jersey Coast Line MP 0.39 (River Draw) Bridge

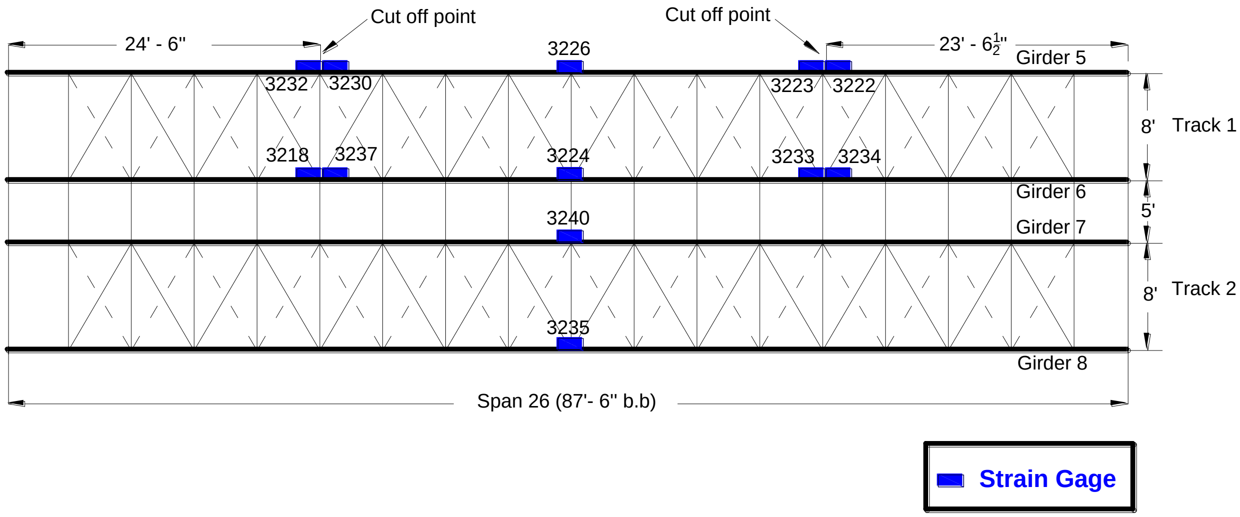


Figure 45. Location of strain transducers installed on various locations in Span 26 of the North Jersey Coast Line MP 0.39 (River Draw) Bridge

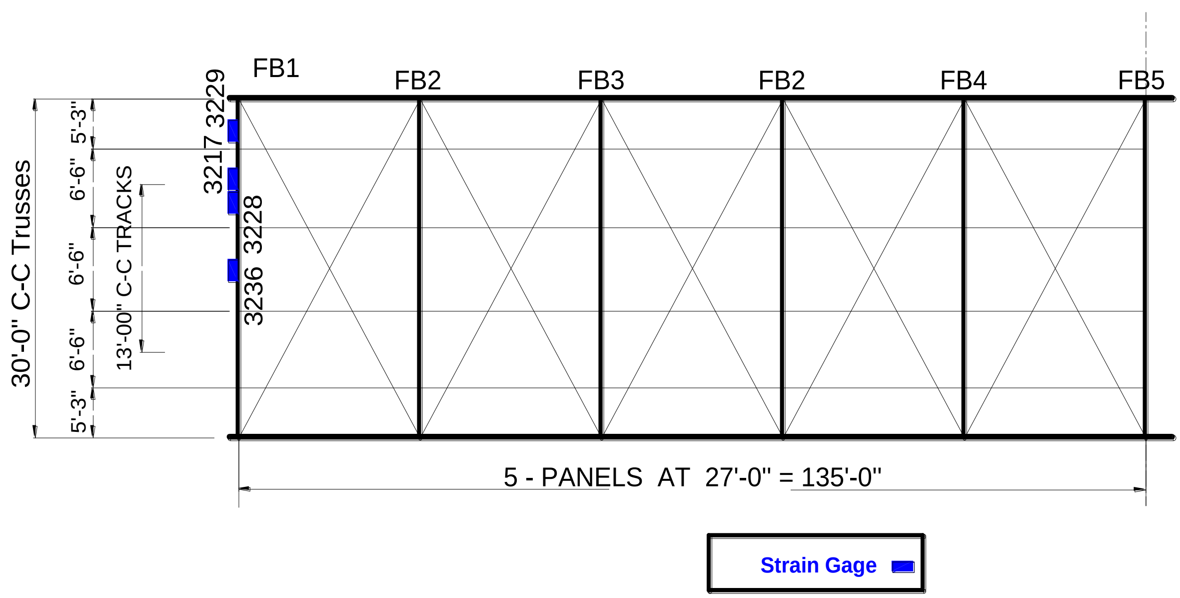
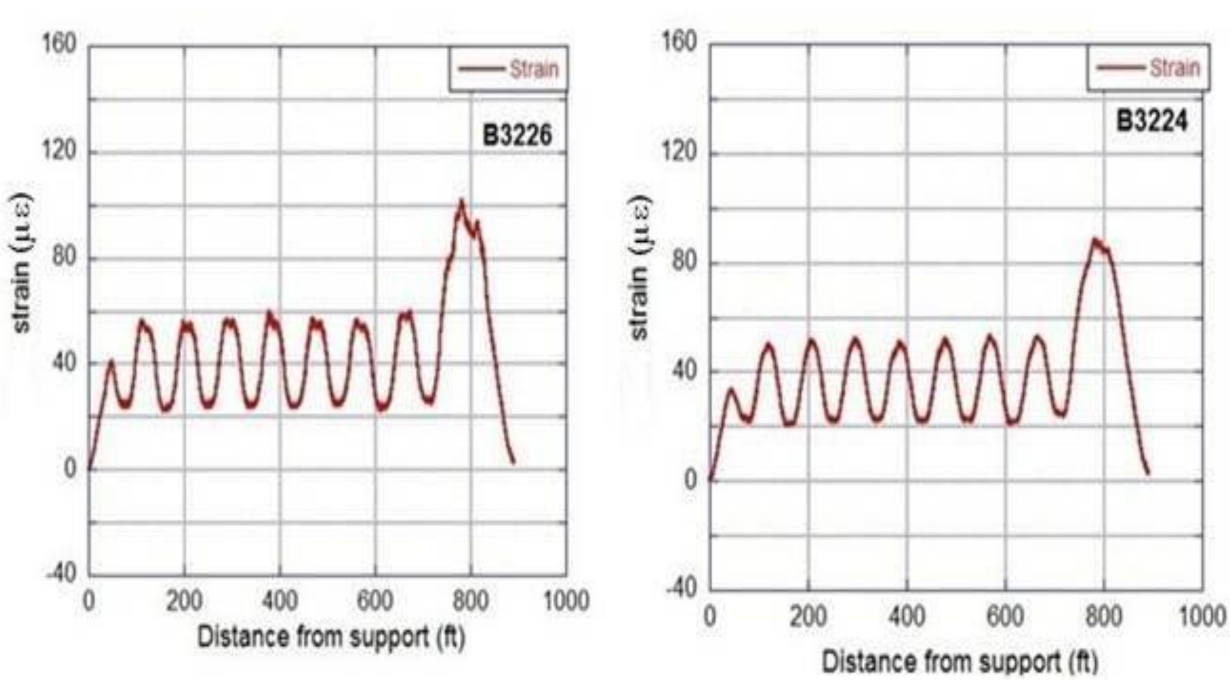


Figure 46. Location of strain transducers in Span 20 of the North Jersey Coast Line MP 0.39 (River Draw) Bridge

Table 20 lists the train information for the tests performed on October 30, 2011. Figure 47 shows the strain data measured at midspan locations during Test Run #1. The maximum strain collected in the field is 106 με, which is equivalent to a maximum stress of 3.1 ksi. Appendix E provides more detailed information on the additional experimental data that was collected from various tests on this bridge.

Table 20 - Train information of tested run on North Jersey Coast Line MP 0.39

Run #	Arrival Time	Direction	Track	Train Symbol	Number of Cars	Locomotive Type	Speed
1	9:38 a.m.	Westbound	1	3227	9	ALP-46A	15.3
2	10:42 a.m.	Westbound	1	3231	8	ALP-46A	17.6
3	12:34 p.m.	Eastbound	2	3244	8	ALP-46A	24.2
4	12:42 p.m.	Westbound	1	3239	9	ALP-46A	12.4
5	1:34 p.m.	Eastbound	2	3248	9	ALP-46	31.7
6	1:40 p.m.	Westbound	1	3243	6	ALP-46A	18.3



(a) *(b)*

Figure 47. Strain data measured during Test Run #1 at Strain Transducer No.*(a)* B3226, *(b)* 3224 on the North Jersey Coast Line MP 0.39 (River Draw) Bridge

Summary of Field-Testing Performed on Selected Bridges

All five selected bridges were tested under passenger trains loading except HX Drawbridge Bridge, which was also tested under 286-kips rail car loading. The maximum strain collected in the field for all five bridges are summarized in Table 21. As shown in Table 21, the maximum measured strain is 261με, collected from the sensor located at midspan of the north girder under Track 2, for the HX Drawbridge Bridge and under 286-kips rail-car loading. The maximum strain collected under passenger train loading is 161 με, collected from the second cutoff of Girder 8 of Span 2 from Raritan Valley Line MP 31.15 Bridge.

Table 21 - Measured maximum strain for selected bridges

Bridge	**Maximum Strain Measured (με)**	**Location of Sensor**
Mainline MP 15.95	109	Floor beam (first cutoff point, G2–G3)
Mainline MP 15.14	105	Floor beam F472 (midspan)
Bergen County HX Drawbridge	261	North girder under Track 2 (midspan)
Raritan Valley Line MP 31.15	161	Girder 8 of Span 2 (second cutoff)
North Jersey Line MP 0.39	115	Girder 6 of Span 26 (first cutoff)

It is noted that during the field-testing of the Bergen County MP 5.48 (HX Draw) Bridge, both dynamic and static tests were performed. The dynamic tests included using both passenger as well as freight trains. The freight train tests were performed at various speeds between 5 mph and 25 mph (i.e., maximum speed limit for freight trains set by NJ Transit). The static tests included stopping the freight train and locating it over the bridge span in order to produce maximum stresses in critical members. These additional tests were made possible given the help and contribution of NJ Transit Engineers, Norfolk Southern (NS) Corp., and various agencies. The freight trains used in the testing had configurations similar to those of a typical 286 kips rail car. Results from the static and dynamic tests using the freight rail car were used in validating the FE model as well as determining the impact factor.

The RT also investigated the section modulus from testing for all five bridges. Taking the Main Line MP 15.95 Bridge as an example, first, the RT found the maximum strain value caused by locomotive cars at various critical locations from the testing data. Then the RT calculated the moment envelope using simple beam analysis method shown in Figure 48 to get the corresponding moment at corresponding critical locations. For this bridge, the loaded length is 43.33’ for Girder 2 (center girder) and the boundary condition is taken as fix-fix. The carload is PL-42 locomotive. Then we calculated the section modulus at each critical location in Table 22.

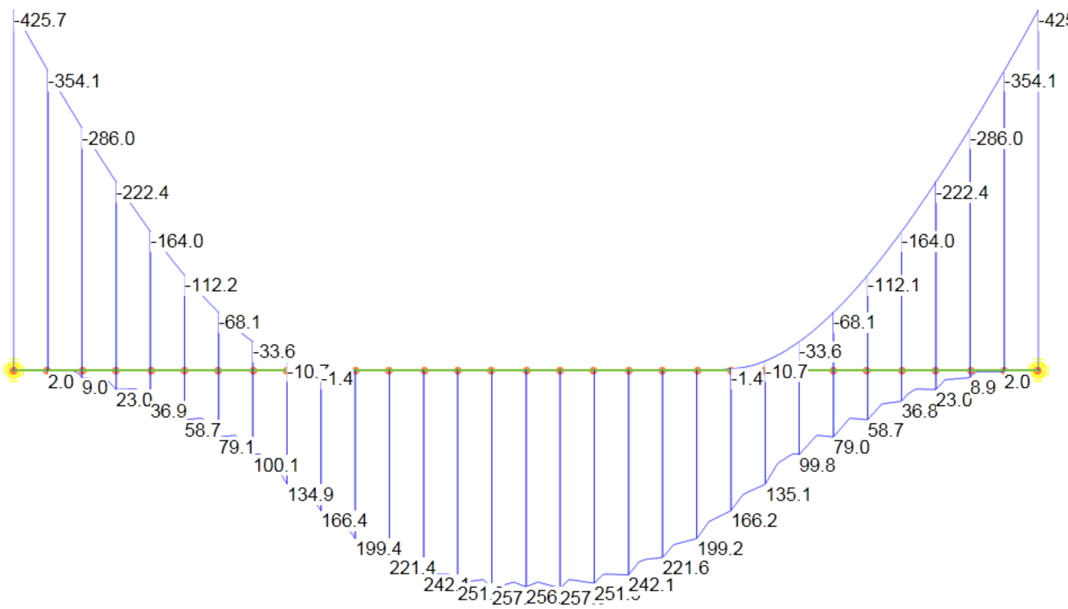


Figure 48. Moment envelope diagram for Main Line MP 15.95 (per girder)

Table 22 - Section modulus calculation using testing data for Main Line MP 15.95

Main Line MP 15.95			
Section	Calculated maximum moment (k-ft)	Maximum strain from testing (με)	Calculated section modulus from testing (in^3)
G2 Sec. 1, (8.06' from support)	79.1	18.0 (B2049 in Run 2)	1818.4
G2 Sec. 2, (10.56' from support)	128.1	25.0 (B2045 in Run 2)	2304.6
G2 Sec. 3, (13.812' from support)	191.2	28.0 (B2491 in Run 2)	2824.9
G2 Sec. 4, (midspan)	256.3	32.0 (B2487 in Run 2)	3313.6

After obtaining the section moduli from testing, they were compared with the different types of section moduli from the inspection reports. As shown in Table 23, “As Built / Inspected (Gross)” and “As Built / Inspected (Net)” are from the inspection reports. It was observed that the calculated section modulus values from field-testing are close to those from the “As Built / Inspected (Net)”. The differences are below 5%. Therefore, the section modulus from the inspection report reflected the real condition for this bridge. The RT used the “As Built/Inspected (net)” value in the FE model.

Table 23 – Summary of section calculation of Main Line MP 15.95

Main Line MP 15.95				
Section modulus (in3)	As Built/Inspected (Gross)	As Built/Inspected (net) (1)	Calculated value from testing (2)	Percent Difference [(1)-(2)]/(2)
G2 Sec. 1, (8.06' from support)	2403.0	1878.5	1818.4	3.31%
G2 Sec. 2, (10.56' from support)	3025.5	2408.6	2304.6	4.51%
G2 Sec. 3, (13.812' from support)	3648.9	2940.5	2824.9	4.09%
G2 Sec. 4, (midspan)	4211.8	3420.9	3313.6	3.24%

Similarly, for other four bridges, the RT calculated the section modulus and compared with the information provided in inspection reports. The results are listed from Table 24 to Table 31.

For the Main Line MP 15.14 Bridge, the boundary condition is also set as “fix-fix”. The locomotive car used in this analysis is F40PH-2CAT. Table 24 shows the calculation of section modulus. From Table 25, the calculated section modulus is slightly higher than the “As Built/Inspected (net)”, which means the inspection reports underestimated the condition of the girders. In the FE model, the RT utilized the calculated section modulus to reflect real condition of the bridge.

Table 24 - Section modulus calculation using testing data for Main Line MP 15.14

Main Line MP 15.14			
Section	Calculated maximum moment (k-ft)	Maximum strain from testing ($\mu\varepsilon$)	Calculated section modulus from testing (in^3)
G29 Sec. 1, (midspan)	82.3	28.0 (B2447 in Run 2)	1357.2
G38 Sec. 4, (midspan)	364.5	45.0 (B2457 in Run 2)	3278.4
G28 Sec. 9, (midspan)	364.5	25.0 (B2455 in Run 2)	6032.3

Table 25 - Summary of section calculation of Main Line MP 15.14

Main Line MP 15.14				
Section modulus (in3)	As-Built/Inspected (Gross)	As Built/Inspected (net) (1)	Calculated value from testing (2)	Percent Difference [(1)-(2)]/(2)
G29 Sec. 1, (midspan)	1449.0	1221.0	1357.2	-10.04%
G38 Sec. 4, (midspan)	3735.0	3077.0	3278.4	-6.14%
G28 Sec. 9, (midspan)	6800.0	5635.0	6032.3	-6.59%

For the Bergen County Line MP 5.48, the analysis applied 286 kips railcar load. The Boundary condition is simple supported. Table 26 shows the calculation of section modulus. From Table 27, the section modulus at section 1 is almost the same as the value that NJ Transit provided while for midspan, the calculated value is higher than the value from inspection reports.

Table 26 - Section modulus calculation using testing data for Bergen County Line MP 5.48

Bergen County Line MP 5.48			
Section	Calculated maximum moment (k-ft)	Maximum strain from testing ($\mu\varepsilon$)	Calculated section modulus from testing (in^3)
G8 Sec. 1, Span2 (2.83' from support)	786.4	208.0 (B2984 in Run 11)	1564.4
G8 Sec. 3 (midspan), Span2	1552.7	250.0 (B2986 in Run 11)	2735.5

Table 27 - Summary of section calculation of Bergen County Line MP 5.48

Bergen County Line MP 5.48						
Section modulus (in3)	As Built (Gross)	As Built (net)	As inspected (Gross)	As inspected (net)	NJ Transit provided	Calculated value from testing
60' spans Sec. 1, (7.17' from support)	1647.6	1413.6	907.6	683.2	1589.0	1564.4
60' spans Sec. 3 (midspan), Span2	2695.8	2267.5	1975.7	1542.2	N/A	2570.0

For Raritan Valley Line MP 31.15, the traffic load is PL-42 locomotive car in the analysis and the boundary condition is simple supported. From Table 29 the calculated section moduli are close to the “As inspected (Gross)” value. This means that the inspection report has underestimate the load carrying capacity of the girders. However, in the FE model, in order to get good correlation with the field testing, the RT still used the “As Built (net)” section modulus except section 1 (used calculated value) from inspection report and adjusted the boundary condition slightly to reflect the real condition in the field (please refer to the explanation about boundary condition below and Figure 50 for more detail).

Table 28 - Section modulus calculation using testing data for Raritan Valley Line MP 31.15

Raritan Valley Line MP 31.15			
Section	Calculated maximum moment (k-ft)	Maximum strain from testing ($\mu\varepsilon$)	Calculated section modulus from testing (in^3)
G8 Sec. 1, Span2 (2.83' from support)	166.4	100.0 (B2984 in Run 11)	688.3
G8 Sec. 2, Span2 (7.07' from support)	352.5	150.0 (B2983 in Run 11)	972.4
G8 Sec. 3 (midspan), Span2	490.3	150.0 (B2986 in Run 11)	1352.4

Table 29 - Summary of section calculation of Raritan Valley Line MP 31.15

Raritan Valley Line MP 31.15						
Section modulus (in3)	As Built (Gross)	As Built (net)	As inspected (Gross)(1)	As inspected (net)	Calculated value from testing (2)	Percent Difference [(1)-(2)]/(2)
G8 Sec. 1, Span2 (2.83' from support)	851.4	766.0	688.5	621.2	688.3	0.03%
G8 Sec. 2, Span2 (7.07' from support)	1162.5	977.3	1000.8	761.7	972.4	2.92%
G8 Sec. 3 (midspan), Span2	1483.1	1252.4	1320.7	984.3	1352.4	-2.34%

Similarly, for North Jersey Coast line MP 0.39, we can also calculate the section modulus from field-testing in Table 30. From the summary in Table 31, we could also find that the calculated value from field testing at Section B is close to the "As inspected (Gross)" value in inspection report (Bridge inspection and rating, final report, North Jersey Coast Line MP 0.39, second cycle, 1996). In the FE model, the RT still used the "As Built (net)" and adjusted the boundary condition slightly to reflect the real condition in the field.

Table 30 - Section modulus calculation using testing data for North Jersey Coast Line MP 0.39

North Jersey Coast Line MP 0.39			
Section	Calculated maximum moment (k-ft)	Maximum strain from testing ($\mu\varepsilon$)	Calculated section modulus from testing (in^3)
Section A (Midspan)	1254.7	100.0 (B3226 in Run 1)	5192.0
Section B	1124.3	110.0 (B3234 in Run 2)	4229.0

Table 31 - Summary of section calculation of North Jersey Coast Line MP 0.39

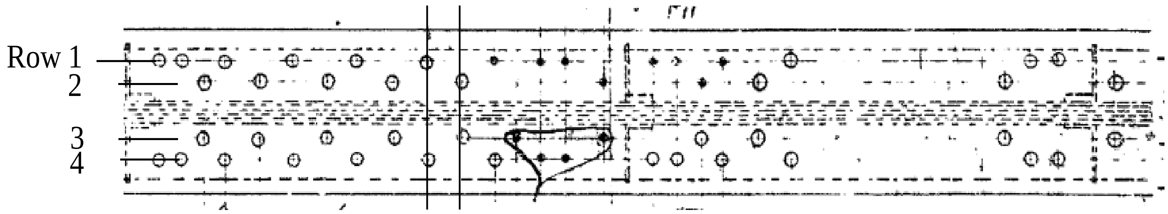
North Jersey Coast Line MP 0.39						
Section modulus (in3)	As Built (Gross)	As Built (net)	As inspected (Gross) (1)	As inspected (net)	Calculated value from testing (2)	Percent Difference [(1)-(2)]/(2)
Section A (Midspan)	5769.0	4924.0	N/A	N/A	5192.0	N/A
Section B (23.15' from support)	4663.0	3938.0	4261.0	3545.0	4229.0	0.76%

From the opinion of the RT, there are three explanations for why would we be getting a difference between various methods of estimating the section modulus:

First, when calculating the net value of section modulus, the inspection report has underestimated the net value. For example in North Jersey Coast Line, although there are 4 rows of rivets (staggered) in the bottom flange of G5 (88' approaching span), but only two rows of rivets appear in the same section as shown in Figure 49. Figure 49 also shows the calculation of net section from inspection report. It shows that the inspection report considers 4 rows of rivets in same section. This will make the net section modulus lower than the real value.

Secondly, the section losses considered in the inspection report is for typical section of same type of girders from different spans, which means that the inspection report overestimates the section losses for a particular girder (tested girders).

Another source of the uncertainty is the different boundary condition assumed in the simple beam analysis and the real bridge. From the field inspection by the Rutgers team, we found that the boundary condition is not exactly simply supported and the girders end are welded to the support over a certain length (see Figure 50). Since the bridge is not exactly simply supported which has unexpected restrain at the support, the real moment in the girder will be lower than the calculated value from simple beam analysis. Therefore the tested strain is lower than the calculated value. However in the FE model, the Rutgers team sets the boundary condition of the bridge model the same as the real boundary condition that reflect the real condition of the bridge. So the FE model is more accurate than the simple beam analysis to simulate the real behavior of the bridge condition under traffic load.



4 row of rivets is not in the same section

Net:

$$I_H = 4 \times 1.69 \times 51.85^2 + 2 \times 3 \times 48.75^2 + 2 \times 1.75 \times 43^2 + 2 \times 1.75 \times 37^2 = 43693$$

$$I_{xN} = 381783 - 43693 = 338090$$

$$S_{xN} = 338090 / 52.69 = 6416$$

Figure 49. Calculation of net section from inspection report(23)

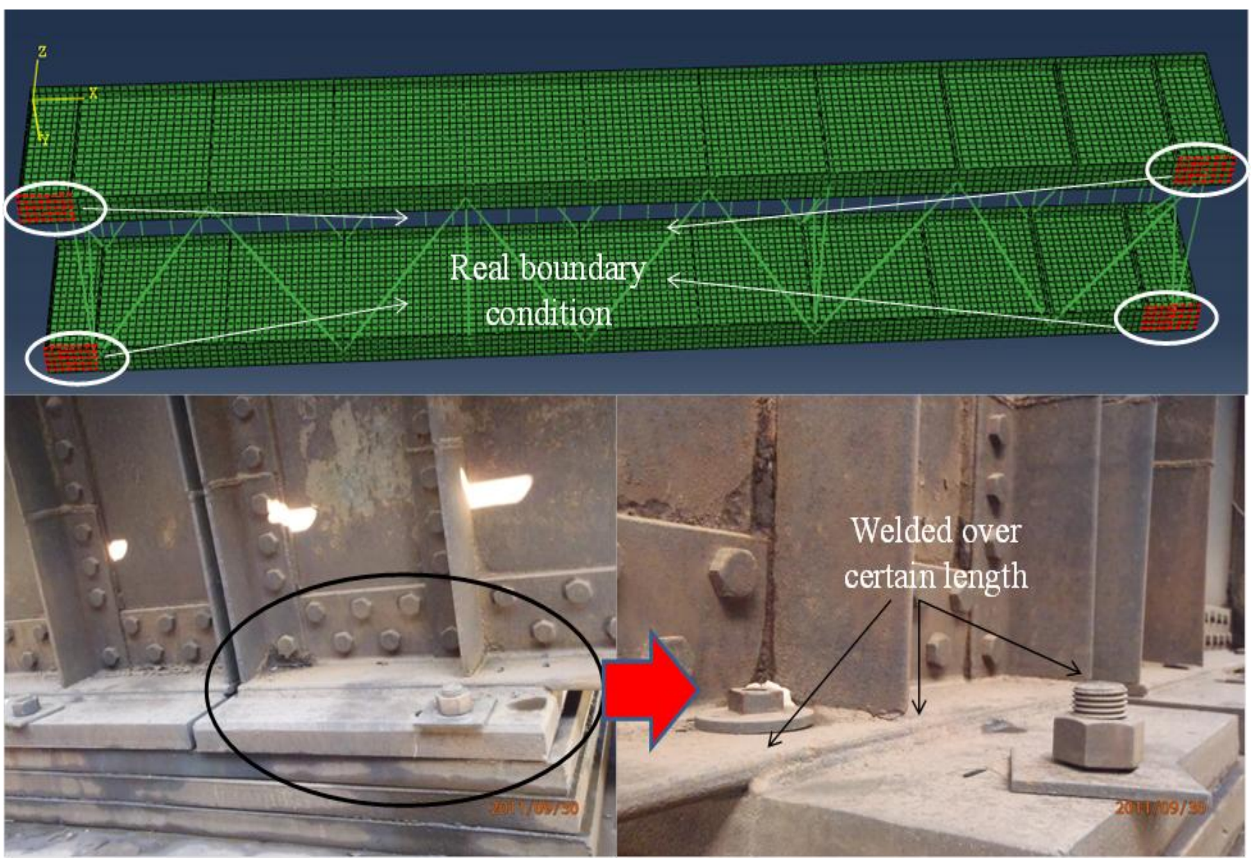


Figure 50. Boundary condition of Raritan Valley Line MP 31.15

Finite Element Bridge Analysis

The five selected bridges were modeled and analyzed using the FE program ABAQUS (Version 6.9.1) to simulate the structural behavior of critical members.[27] The ultimate objective of the detailed analysis is to evaluate more accurately the load rating of the bridge under a typical 286-kips rail car loading. This section illustrates the FE model in ABAQUS of the selected bridges. Figure 51 illustrates an isometric view of the various FE models for the five selected bridges.

To improve the analysis results, various modeling features were considered in the three-dimensional FE model, such as: 1) element types, 2) material behavior, 3) boundary conditions, and 4) interaction between the floor beams and steel girders.

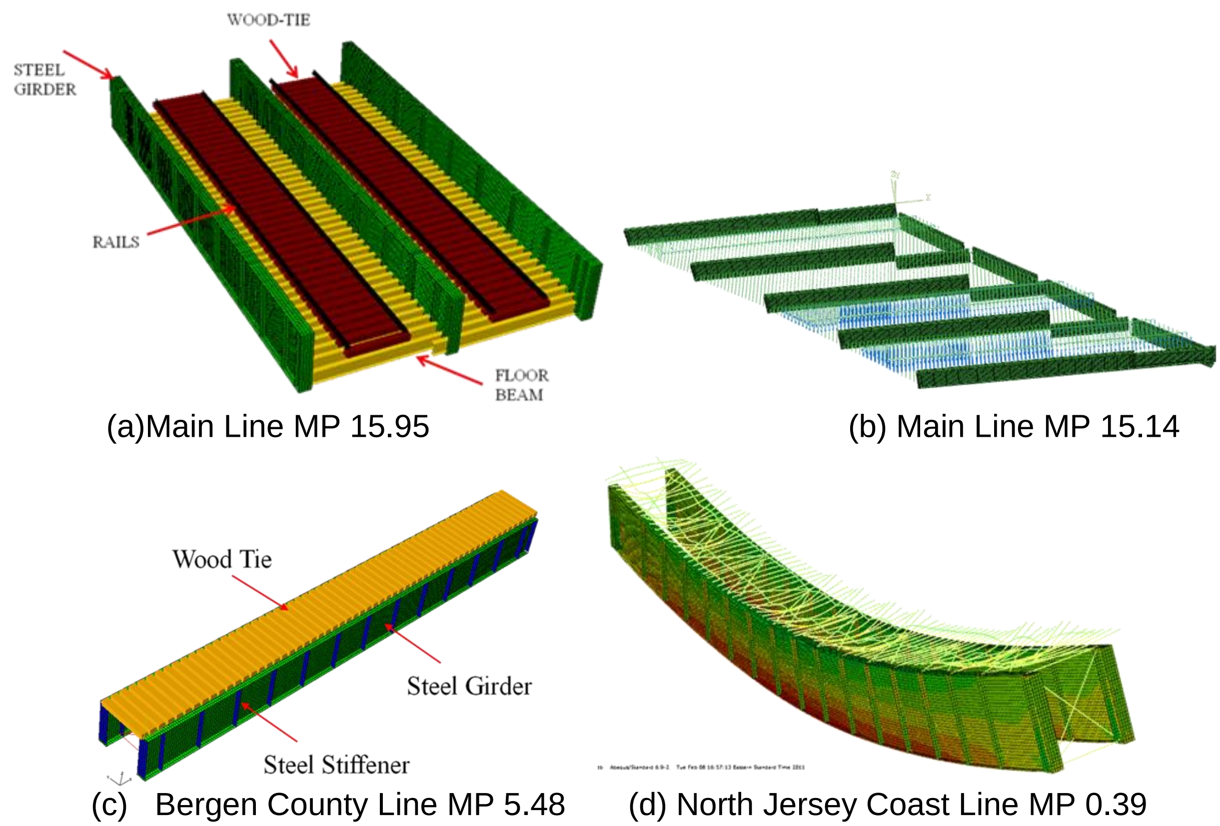


Figure 51. FE Model for four of the selected bridges: (*a*) Main Line MP 15.95, (*b*) Main Line MP 15.14, (*c*) Bergen County Line MP 5.48, (*d*) North Jersey Coast Line MP 0.39

<u>Material Properties</u>

The modulus of elasticity of the steel girder, steel beams and rails, E, and Poisson's Ratio, ν_s, is used as 29,000 ksi and 0.3, respectively. It is noted that the steel girders, beams, and rails are expected to undergo deformation within the elastic range only and, therefore, the inelastic behavior of the steel material was not considered. Material properties for wooden-tie members such as modulus of elasticity, E, and Poisson's Ratio were considered as 1,600 ksi and 0.3, respectively.

Element Selection and Analysis Procedure

The steel girders were modeled by using a four-node shell element (S4). Element type S4 in ABAQUS is a fully integrated, finite-membrane-strain shell element. Simpson's Rule was used to calculate the cross-sectional behavior of the shell elements.(27)

A two-node linear beam element (B31) was selected in the model to simulate the steel floor beams, rails, and wood ties. The element type B31 is a first-order, shear-deformable beam element, which accounts for shear as well as flexural deformations in the analysis.(27)

One type of connector element was also used in the FE analysis model to join two nodes. Connection type JOIN, which forces the position of one node to be the same as the second node, was used to idealize the pin connections. The JOIN type of connector was used to idealize the bold connections between steel girders and floor beams.

As illustrated in Figure 52, JOIN type connectors were also used to simulate the ballast deck between the wood ties and the floor beams in the bridges with ballast decks. Every element of the wood–tie members was connected to the floor beam members to distribute the load uniformly, simulating the role of the ballast deck. JOIN connectors were also used to connect the rail elements to the wood-tie elements

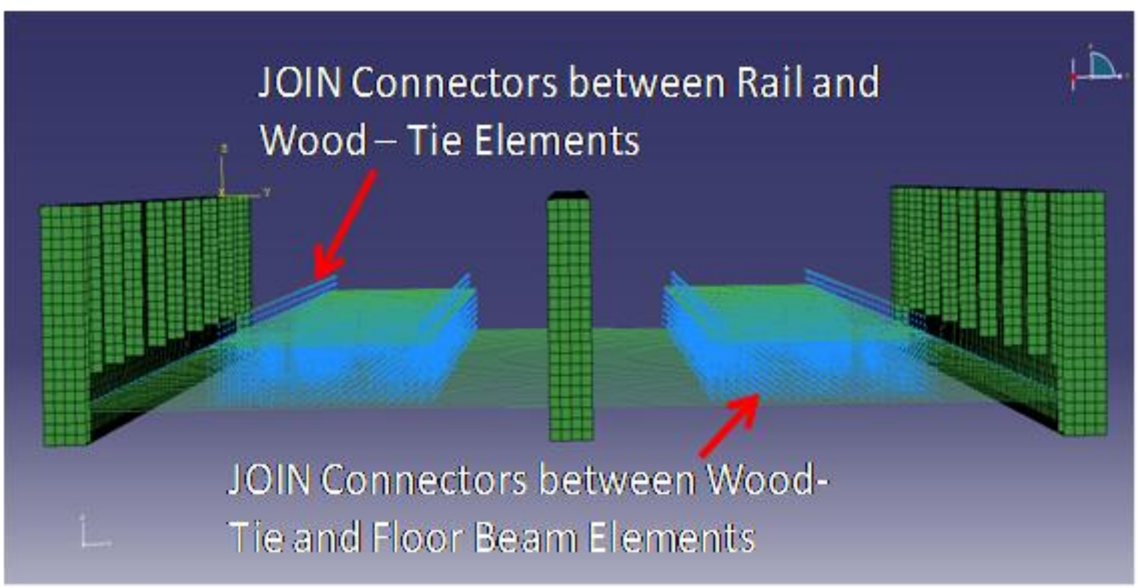


Figure 52. Connectors between different element sets

In the FE model, a set of point loads simulating a rail car was applied on the rail elements. A multiple load case analysis was adopted to apply the rail car loading at various nodes on both tracks of the selected bridge.

The accuracy of the model was verified by comparing the strain and deflection results obtained from the FE analysis and field test data, as explained in the next section.

Model Verification

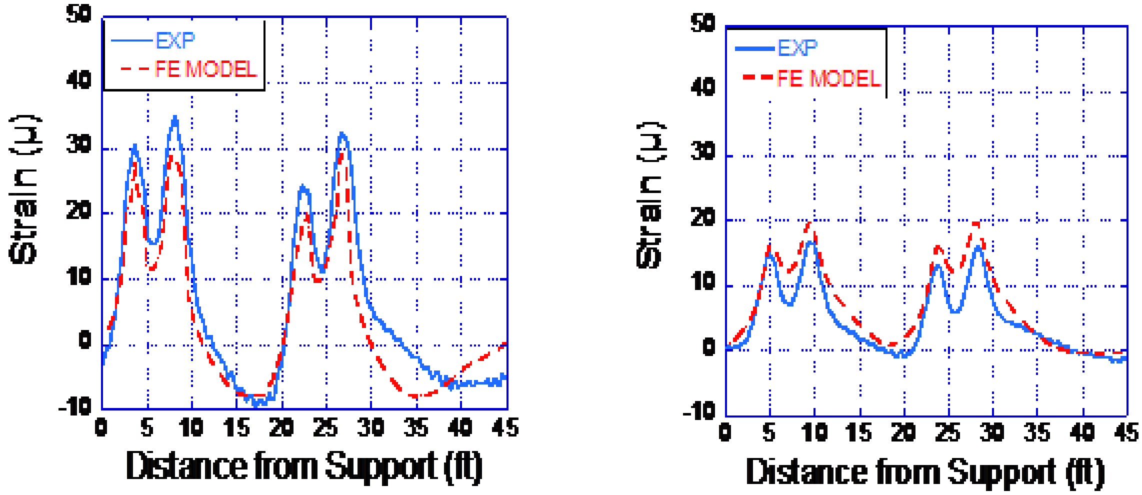
In this section, the verification and calibration was performed for all five bridges, and some adjustments to the model were made if needed to improve the accuracy of the model. Since the verified models will be used in the load rating part under different load scenarios, the maximum structural response under traffic load is the most important issue. Therefore in this part, the difference in the form of percentage between the FE model and field-testing data was computed at the peak value to verify the models as well as the average value and coefficient of variation (COV). The difference between the FE model analysis results and the field test data can be attributed to various reasons, but it is mainly the result of the dynamic impact, the damping effect, the unexpected restraints at member connections and end supports. Additionally, possible small dimension differences between the actual bridge sections and the FE model can be also attributed to the variation between the analysis results and the field test results.

In general for bridges with ballast deck, another reason for having a variation between FE model results and field-testing data for model simulation can be attributed to the idealization of load distribution through the ballast deck. The connectors between wood-tie members and floor beams were modeled in such a way to help distribute the load applied on the rail element. However, in reality, the load is distributed more evenly to the floor beams and girders through the ballast deck.

Verification of the Main Line MP 15.95 Bridge Model

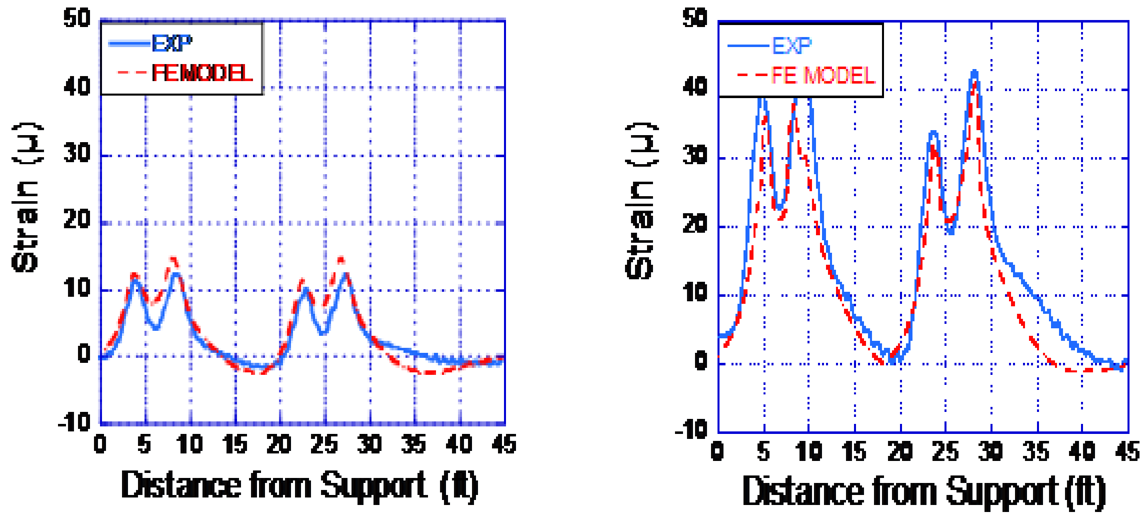
Deflections and strains of the structural elements were recorded from the strain transducers and LDV unit as the tested rail cars passed over the bridge span. The obtained deflection and strain results of the structural members under the rail-car loading were compared with the analysis results.

Figure 53 through Figure 55 show comparison of strain records for Test Run #2, when the rail car travels from Hawthorne to Paterson. The horizontal axis shows the location of the rail-car front axle moving from the west support of the span (as the designated direction in the inspection reports). The same analysis was carried out for Test Run #3, as shown in Figure 56 through Figure 59, when the rail car travels in the opposite direction from Paterson to Hawthorne. The horizontal axis shows the location of the rail car front axle moving from the east support of the span (as the designated direction in the Inspection reports).



(a) (b)

Figure 53. Comparison of strain results between FE analysis and field test data in Test Run#2 at *(a)* Sensor 2049 and *(b)* Sensor 2046, for the Main Line MP 15.95 Bridge



(a) (b)

Figure 54. Comparison of strain results between FE analysis and field test data in Test Run#2, at *(a)* Sensor 2050 and *(b)* Sensor 2045, for the Main Line MP 15.95 Bridge

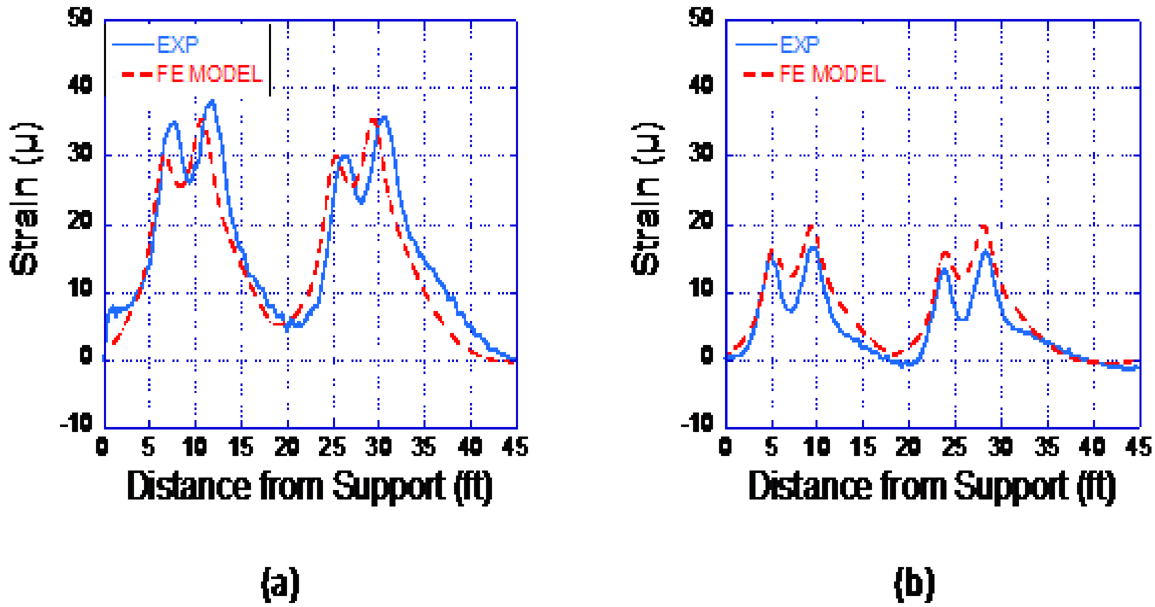


Figure 55. Comparison of strain results between FE analysis and field test data in Test Run #2, at *(a)* Sensor 2487 and *(b)* Sensor 2491, for the Main Line MP 15.95 Bridge

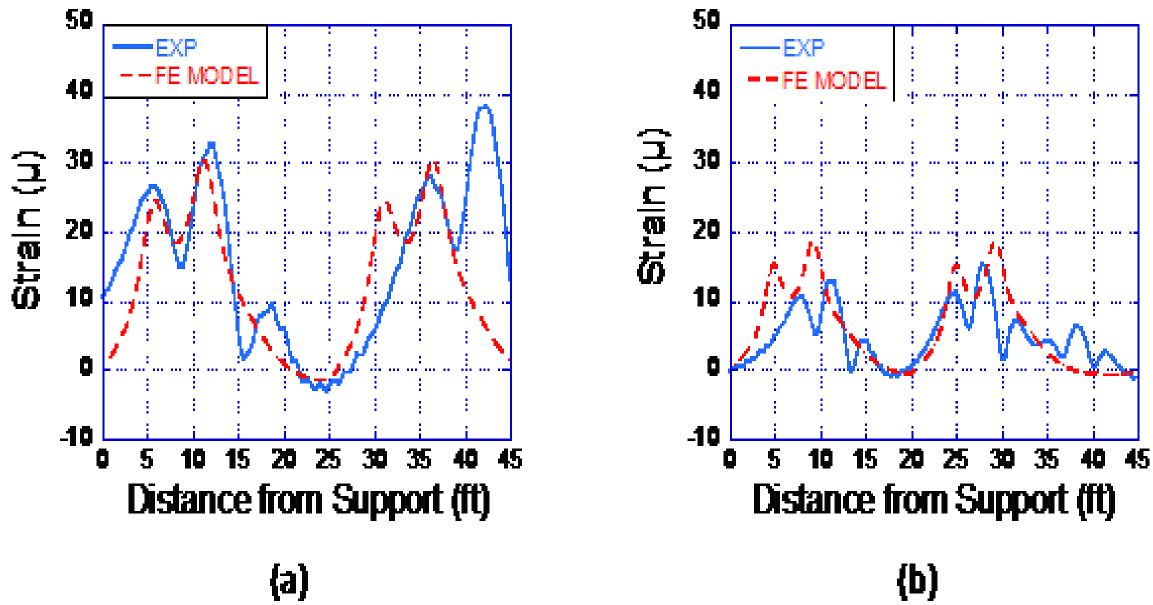


Figure 56. Comparison of strain results between FE analysis and field test data in Test Run #3, at *(a)* Sensor 2045 and *(b)* Sensor 2046, for the Main Line MP 15.95 Bridge

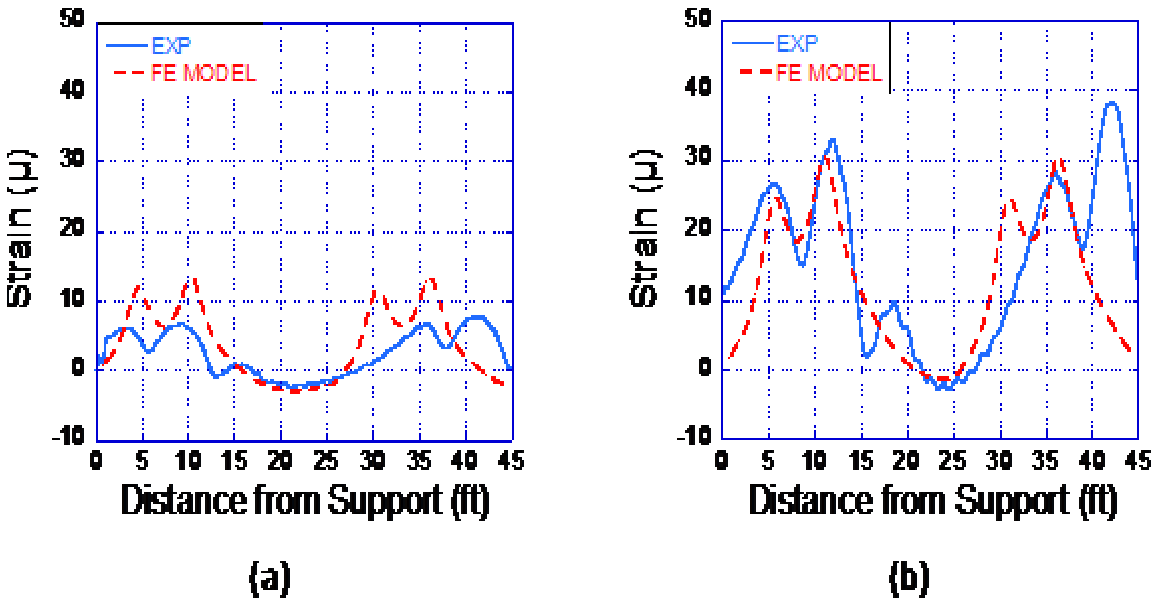


Figure 57. Comparison of strain results between FE analysis and field test data in Test Run#3, at *(a)* Sensor 2049 and *(b)* Sensor 2050, for the Main Line MP 15.95 Bridge

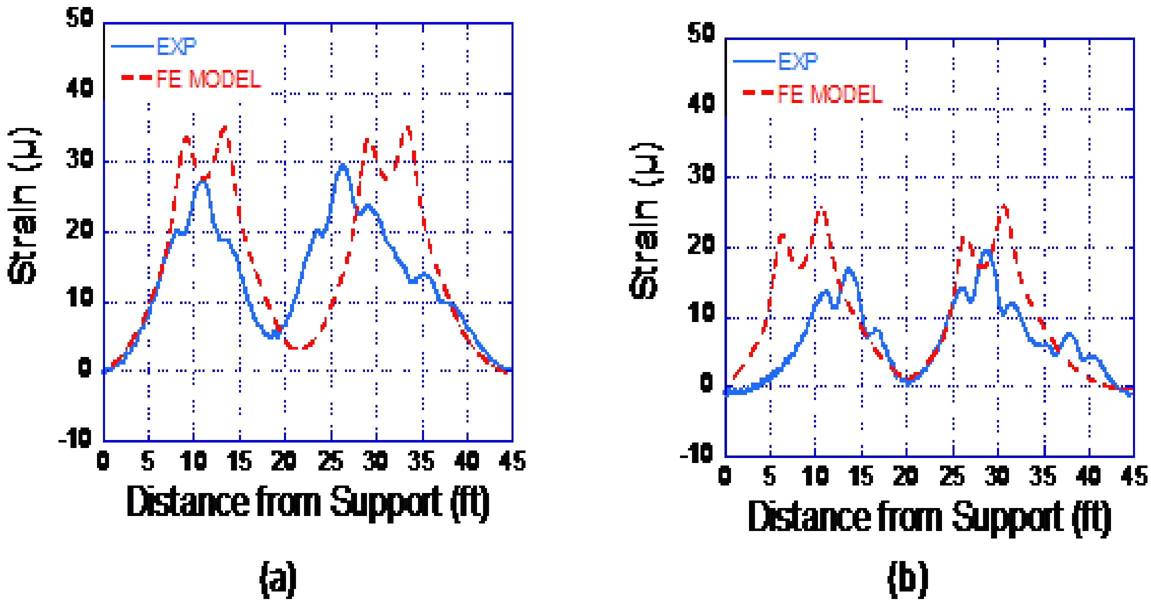


Figure 58. Comparison of strain results between FE analysis and field test data in Test Run#3, at *(a)* Sensor 2487 and *(b)* Sensor 2490, for the Main Line MP 15.95 Bridge

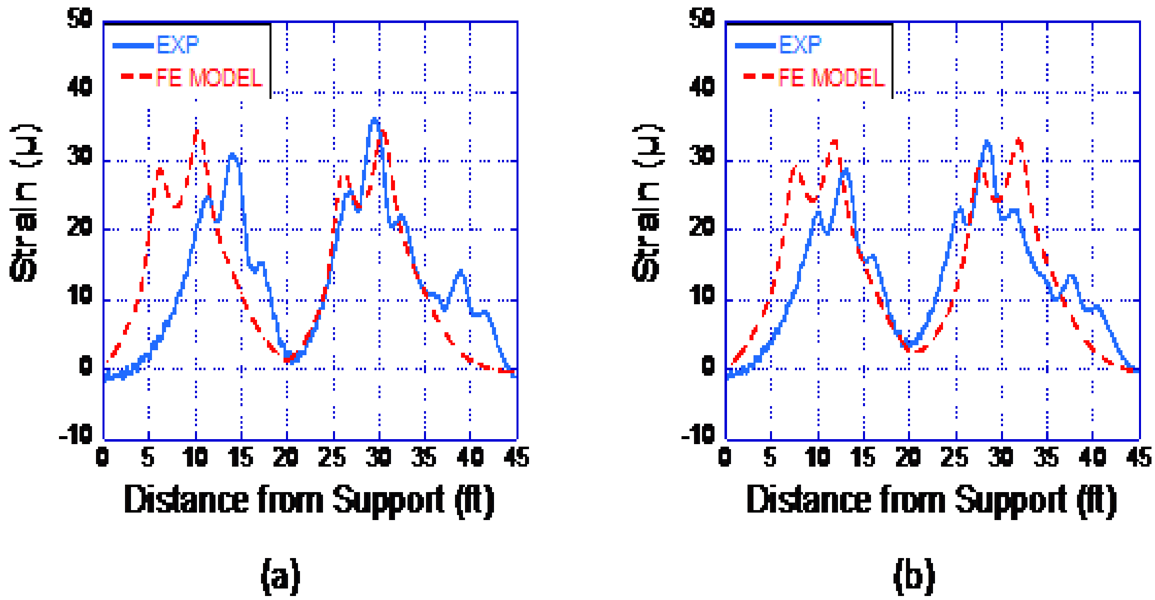


Figure 59. Comparison of strain results between FE analysis and field test data in Test Run#3, at *(a)* Sensor 2491 and *(b)* Sensor 2493, for the Main Line MP 15.95 Bridge

Figure 60 shows a comparison of deflection results between FE model and field data at the midspan for both Cases No.1 and No. 2. The horizontal axis shows the front axle distance from the support in the traveling direction.

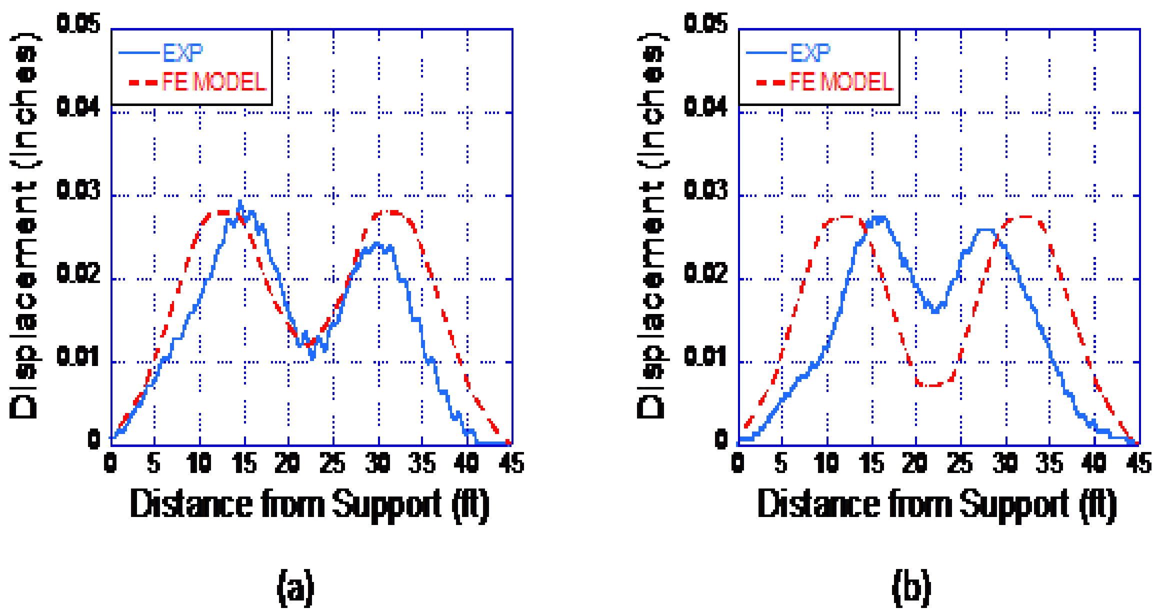


Figure 60. Comparison of deflection results between FE analysis and field test data, for *(a)* test run #1 and *(b)* test run#2, for the Main Line MP 15.95 Bridge

Table 32 shows a comparison of the percentage difference between results from FE model and those from the field-testing. It can be observed that the average difference in the form of percentage is 8.75% and the coefficient of variation is 92.8%. For this bridge, since the structural response (strain) under running train is relatively low (less than 50 με) compared to other bridges, although the percentage difference is 8.75%, the difference between the FE model and testing data is very low. For example, at the location of B2046, the difference is only 3 με while the percentage difference is 17.6%. This difference can also be affected by the accuracy of the testing equipment. Also, the dynamic impact can be part of the difference. Overall, it can be seen that the FE model results exhibited good agreement with the testing results under the same rail car loading.

Table 32 - Comparison of results from FE model and field-testing for the Main Line MP 15.95 Bridge

Comparison		FE value	Testing value	Difference	Percentage Difference
B2049	Peak	30 με	35 με	5 με	-14.30%
B2046	Peak	20 με	17 με	3 με	17.65%
B2050	Peak	15 με	13 με	2 με	15.38%
B2045	Peak	42 με	41 με	1 με	2.44%
B2487	Peak	38 με	37 με	1 με	2.70%
B2491	Peak	20 με	17 με	3 με	17.65%
Deflection (Run1)	Peak	0.028 in	0.028 in	0.000 in	0.00%
Deflection (Run2)	Peak	0.028 in	0.028 in	0.000 in	0.00%
			Average		8.75%

Verification of Main Line MP 15.14 Bridge FE Model

For the Main Line MP 15.14 Bridge, spans 1 and 2 were instrumented with various sensors and modeled using the FE analysis software ABAQUS. Strains of the structural elements were recorded from the strain transducers as the test rail cars passed over the bridge span, respectively. The strains from various structural members obtained under the effect of railcar loading were compared with the analysis results.

Figure 61 shows correlation between FE analysis results and field-testing data at various recorded times. Table 33 shows that the average percentage difference between the FE model and field-testing results is 4.82% and the coefficient of variation is 74.5%. The dynamic impact and the accuracy of the equipment can be part of the reason that caused the difference.

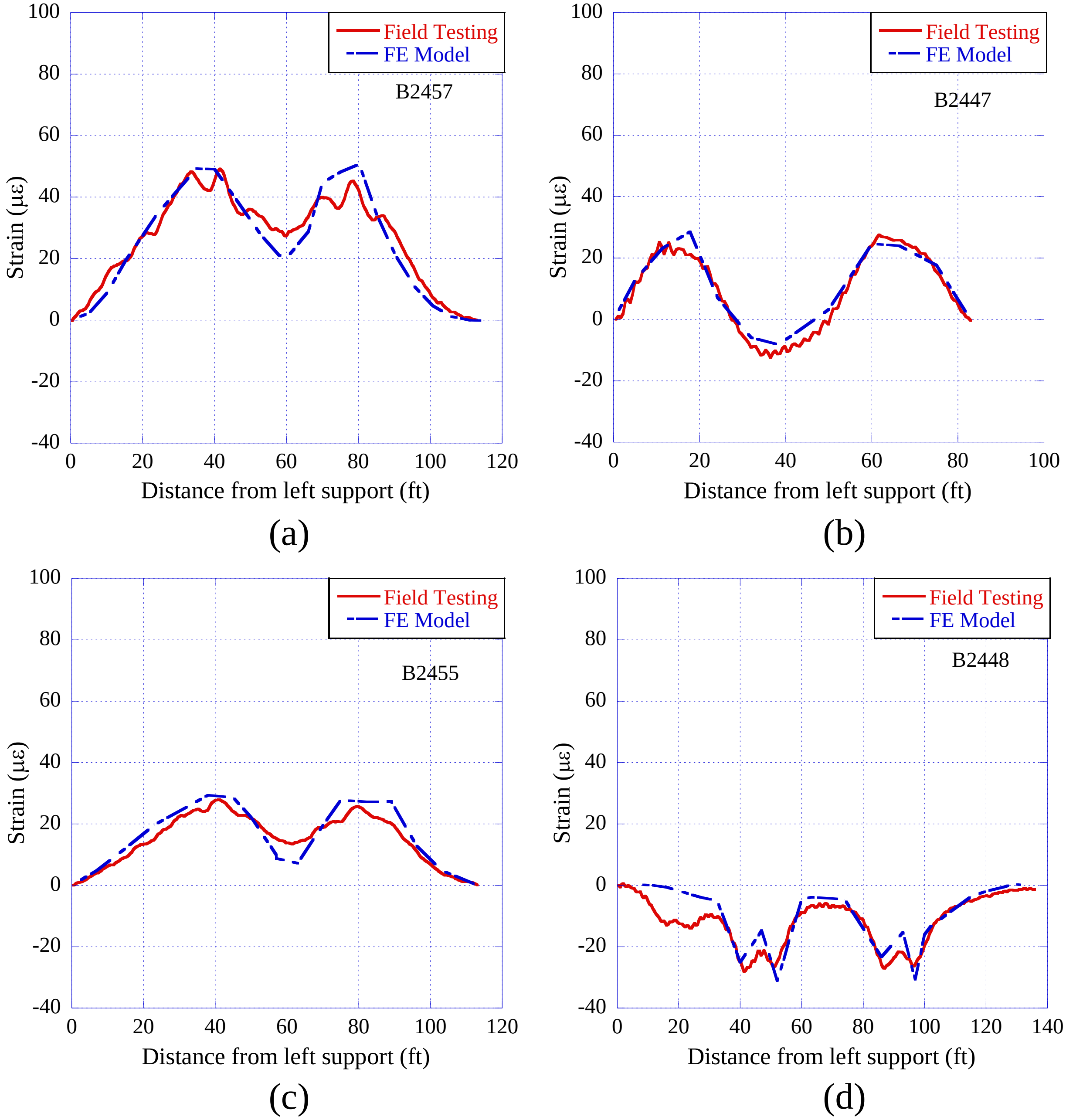


Figure 61. Comparison of strain data collected in field-testing with FE model *(a)* 2457,*(b)* 2447,*(c)* 2455,and *(d)* 2448 for the Main Line MP 15.14 Bridge

Table 33 - Comparison of results from FE model and field-testing for the Main Line MP 15.14 Bridge

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2457	Peak	50	48	2	4.17%
B2447	Peak	28	26	2	7.69%
B2455	Peak	30	30	0	0.00%
B2448	Peak	-29	-27	2	7.41%
			Average	1.5	4.82%

Verification of the Bergen County MP 5.48 (HX Draw) Bridge Model

Strains were recorded from the strain transducers installed on the structural elements as the test rail cars passed over the Bergen County Line MP 5.48 Bridge. The section modulus for each member is based on the updated drawings provided by NJ Transit.

Figure 62 through Figure 69 show the field data collected from strain transducers number B2568, B2564, B2567, B2662, B2563, B2569, B2566, and B2574, respectively, compared with FE analysis results with static testing, as well as dynamic testing under various train speeds. The horizontal axis shows the location of the rail-car front axle from the left support of the span. In the FE model, the loading used was the last three 286-kips freight cars (NS 994198, NS 994782, and NS4671) to simulate the rail car passing through the selected bridge. Table 34 through Table 37 listed the difference between FE model and field-testing in terms of percentage with static testing, as well as dynamic testing under various train speeds.

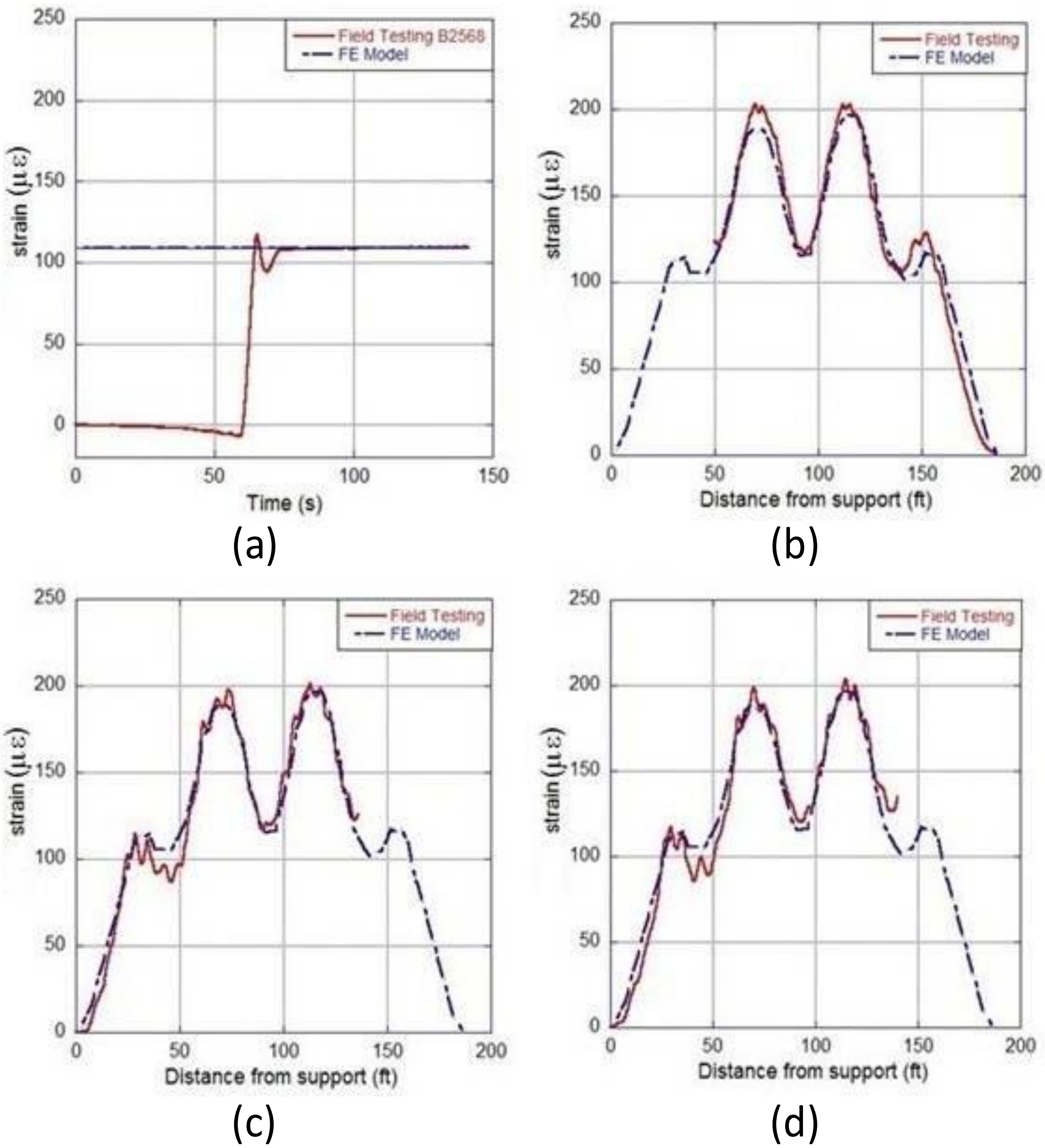


Figure 62. Comparison of strain data collected from Sensor No. B2568 and FE model:*(a)* static testing,*(b)* 10 mph in Run #5,*(c)* 20mph in Run #18, and *(d)*25 mph in Run #20

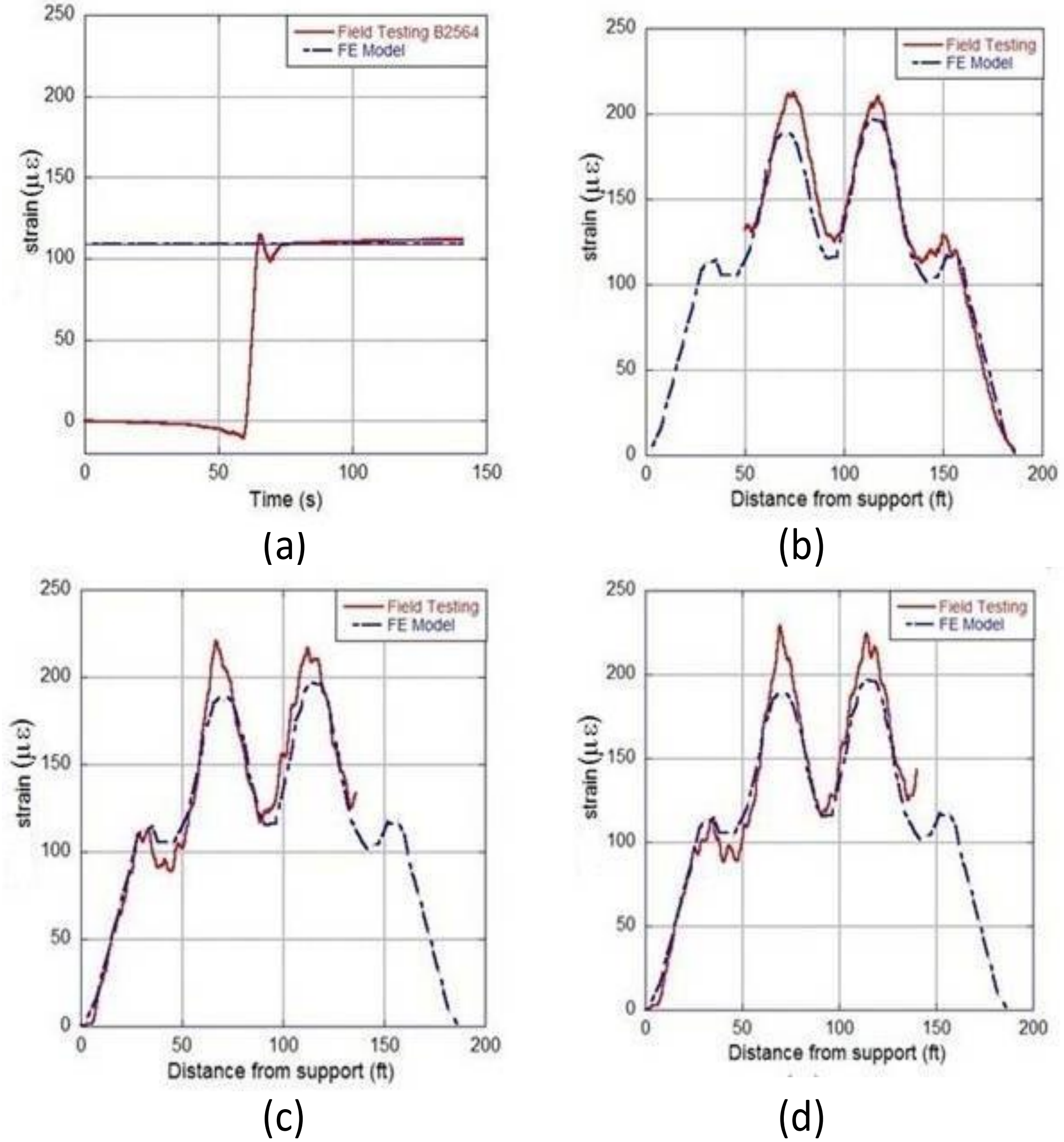


Figure 63. Comparison of strain data collected from Sensor No. B2564 data with FE model:*(a)* static testing,*(b)* 10 mph in Run #5,*(c)* 20 mph in Run #18, and *(d)* 25 mph in Run #20

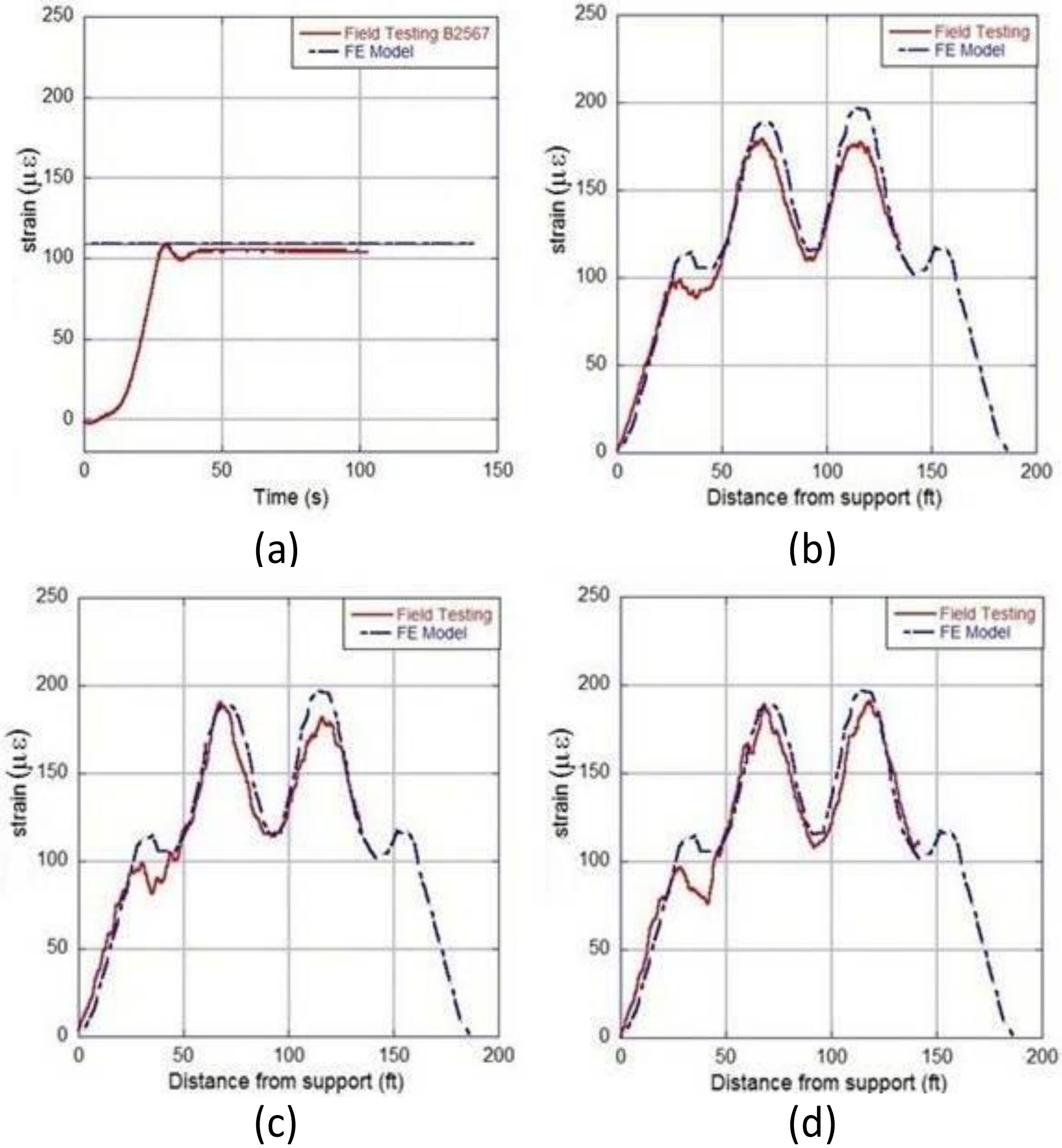


Figure 64. Comparison of strain data collected from Sensor No. B2567 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #30,*(c)* 20mph in Test Run 32, and *(d)* 25 mph in Test Run #34

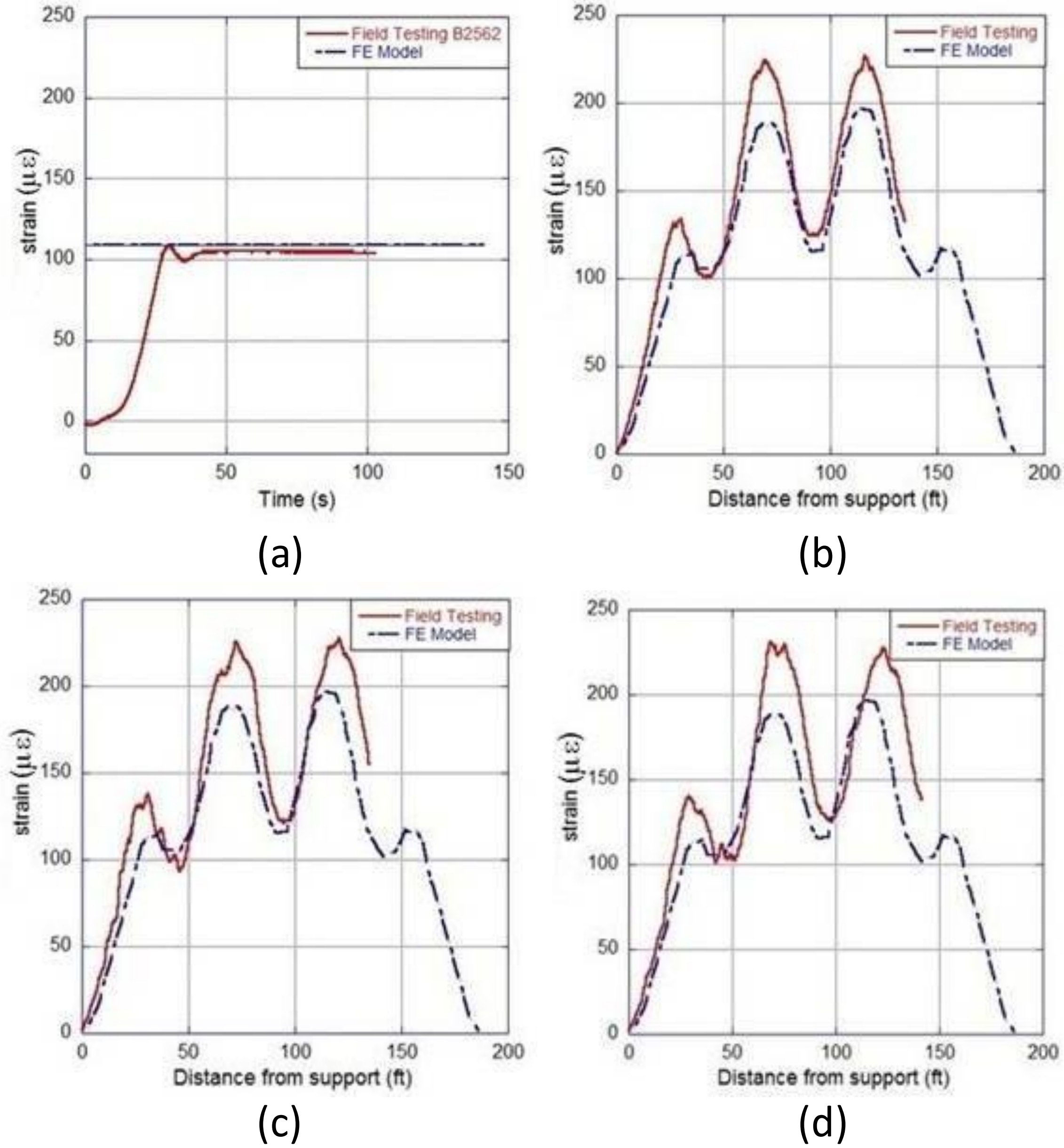


Figure 65. Comparison of strain data collected from Sensor No. B2562 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #30,*(c)* 20mph in Test Run #32, and *(d)* 25 mph in Test Run #34

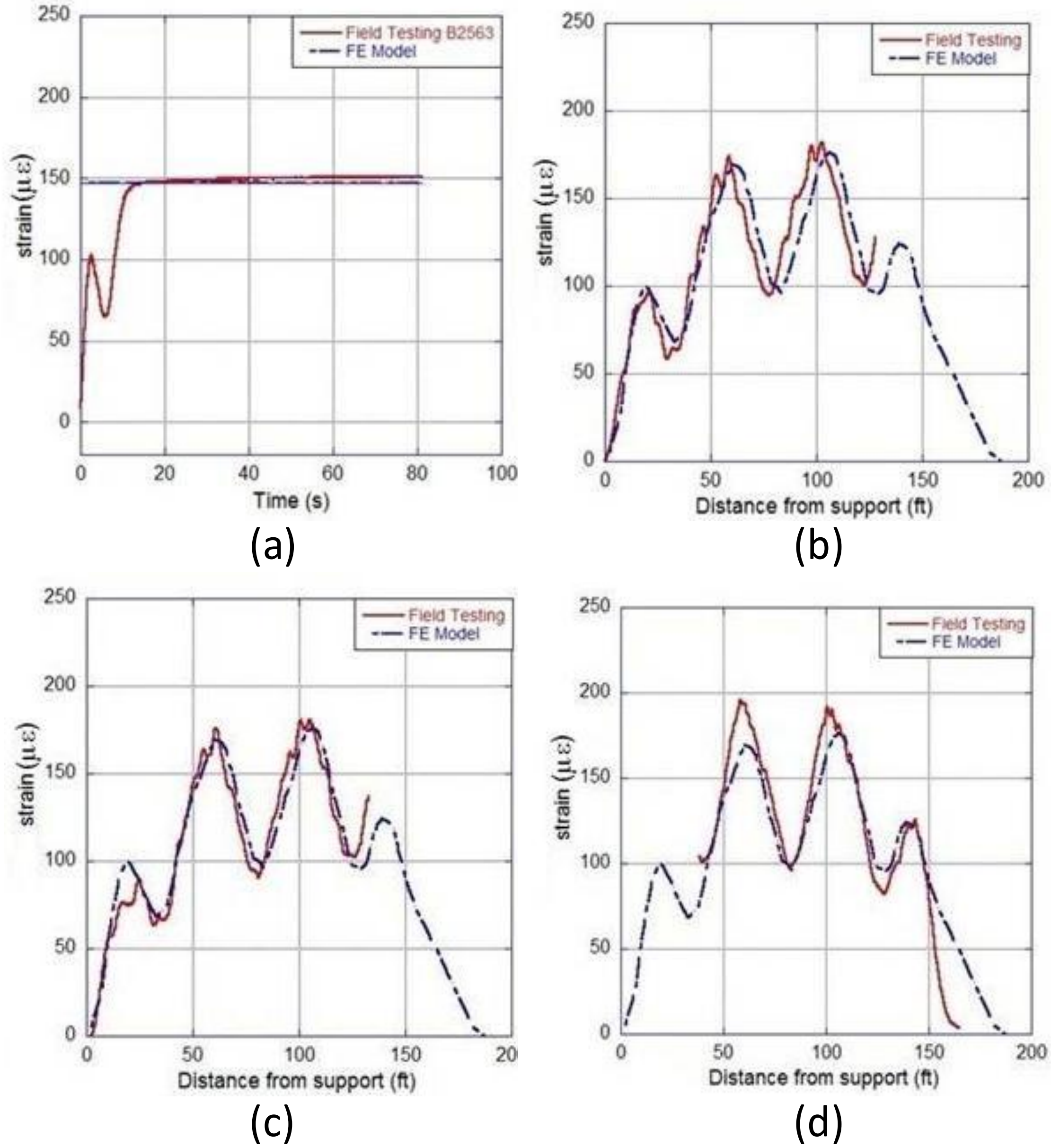


Figure 66. Comparison of strain data collected from Sensor No. B2563 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #5,*(c)* 20mph in Test Run #18, and *(d)* 25 mph in Test Run #15

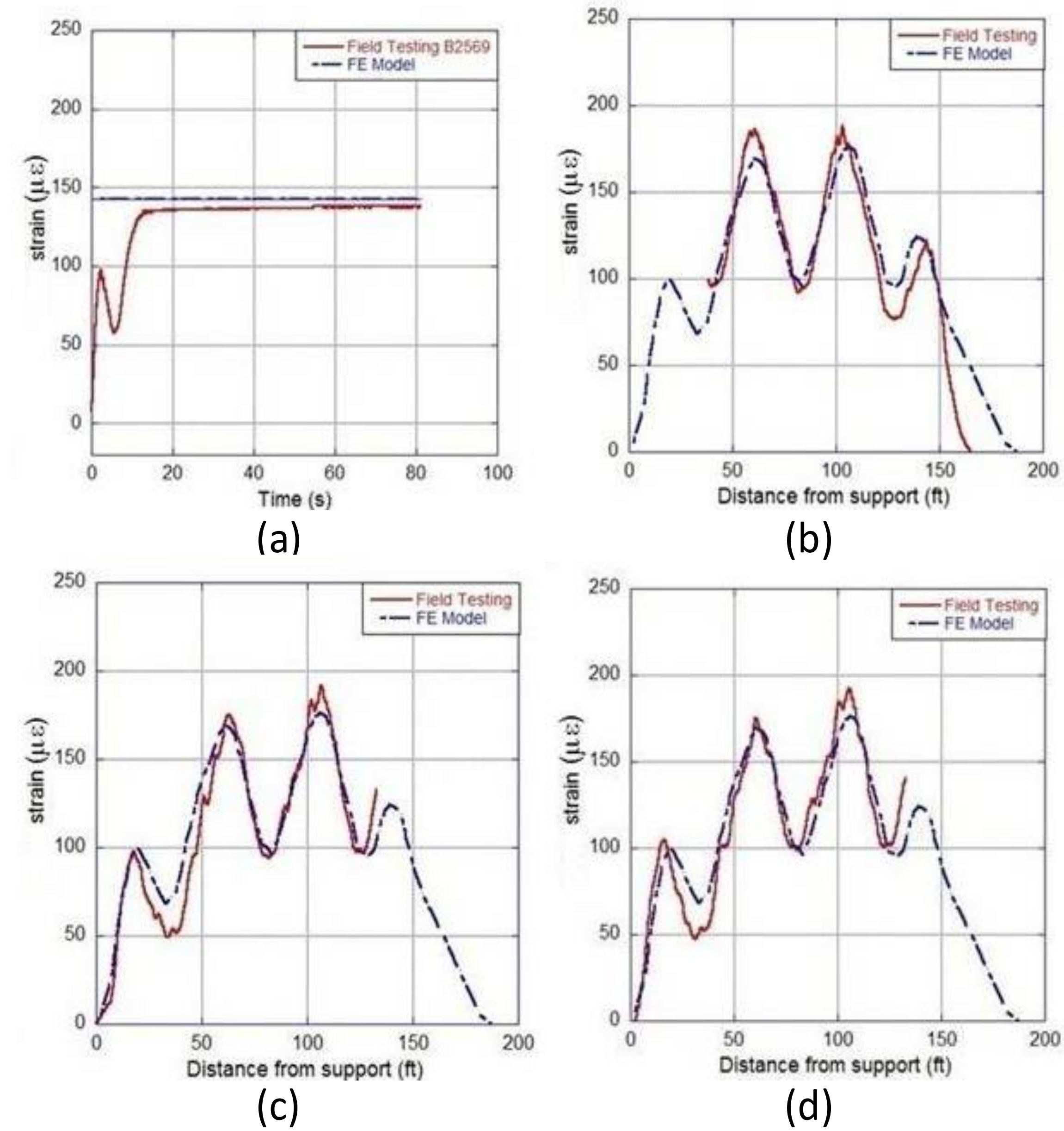


Figure 67. Comparison of strain data collected from Sensor No. B2569 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #5,*(c)* 20mph in Test Run #18, and *(d)* 25 mph in Test Run #15

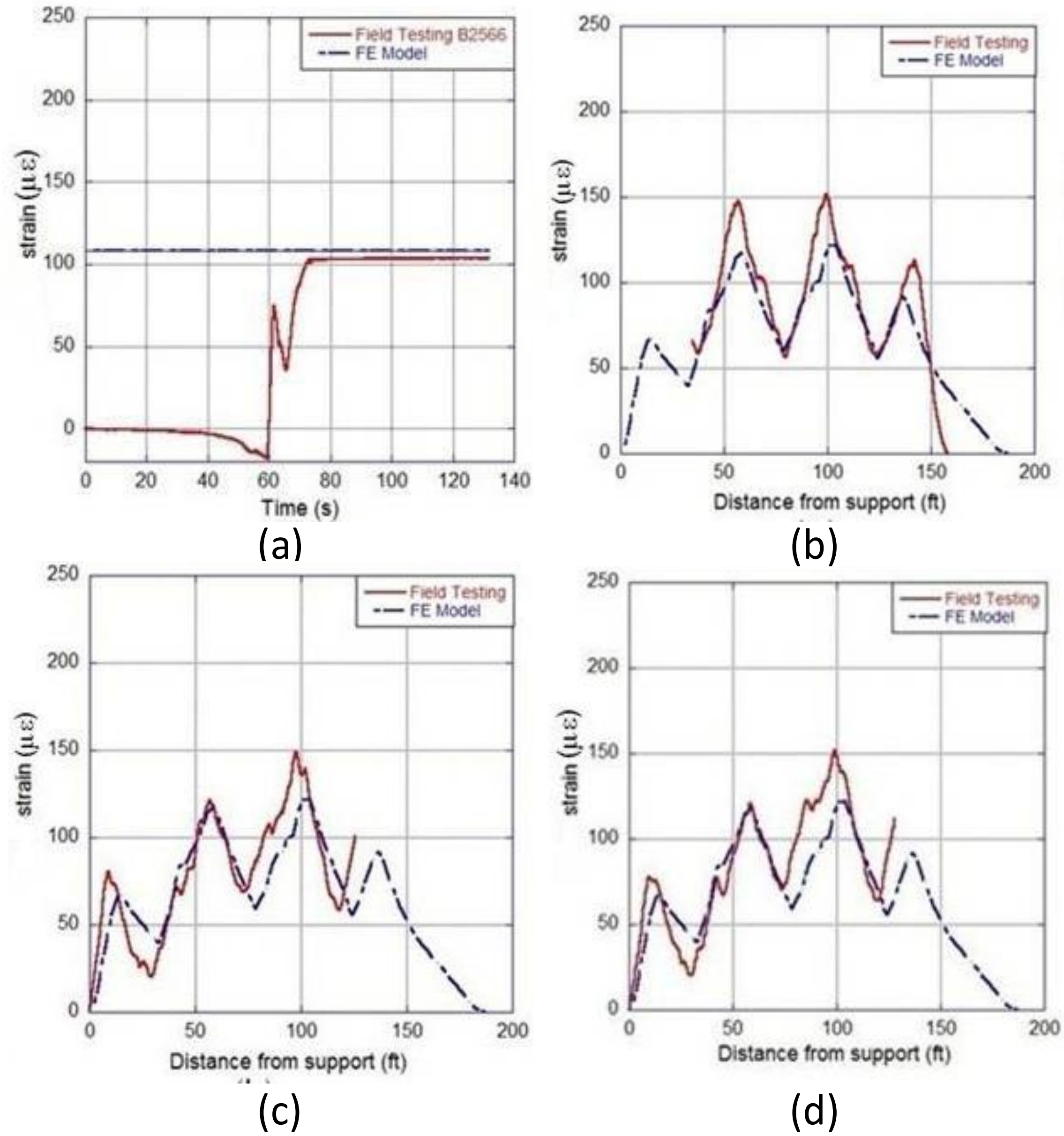


Figure 68. Comparison of strain data collected from Sensor No. B2566 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #5,*(c)* 20 mph in Test Run #18, and *(d)* 25mph in Test Run #20

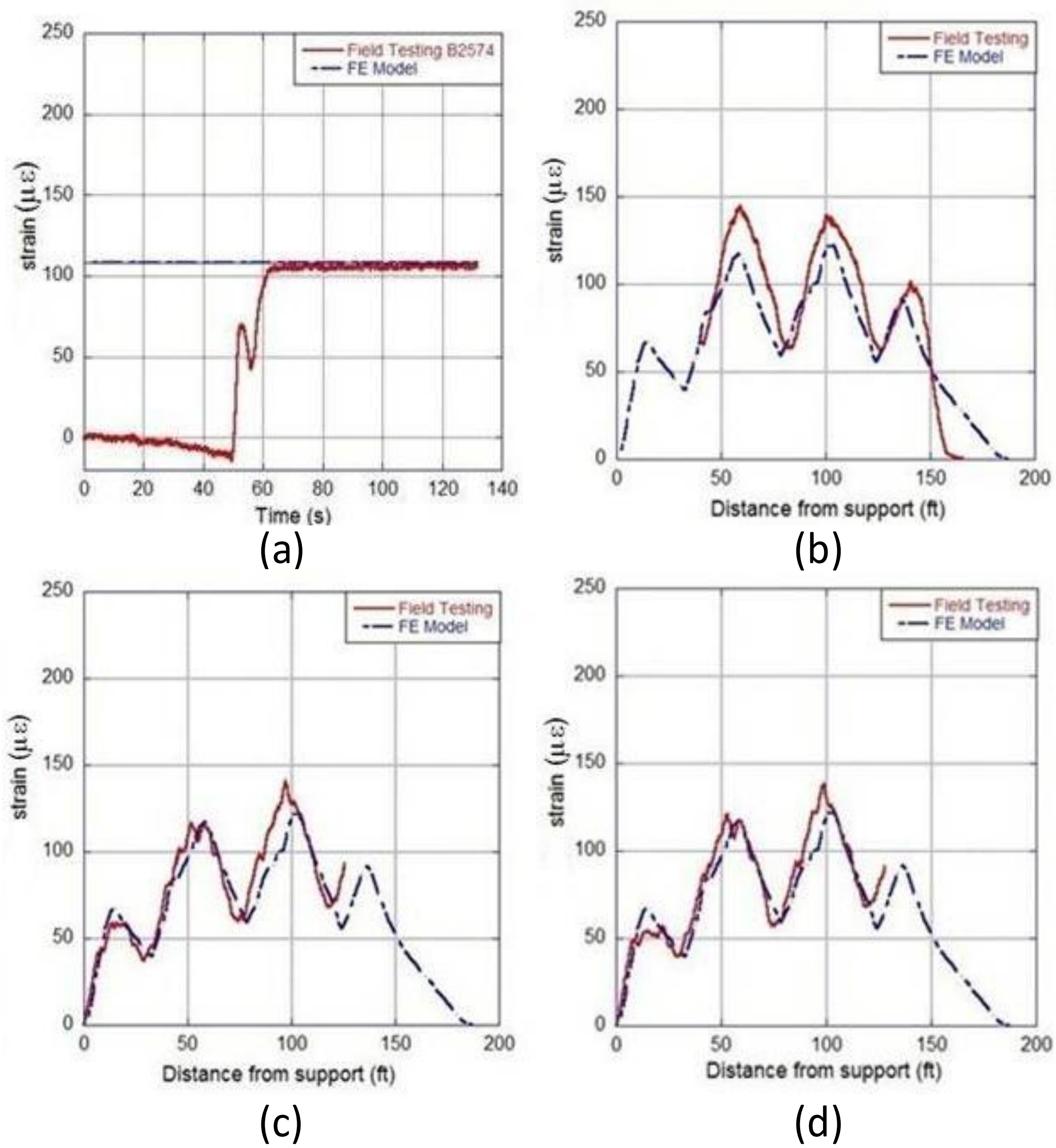


Figure 69. Comparison of strain data collected from sensor No. B2574 data with FE model: *(a)* static testing,*(b)* 10 mph in Test Run #5,*(c)* 20 mph in Test Run #18, and *(d)* 25 mph in Test Run #20

Table 34 shows that the maximum percentage difference between FE model and the static testing data is 5.7% which means that the results from the FE model exhibited very good correlation with those obtained from the static field testing (average percentage difference 2.7% and coefficient of variation 92.6%). Moreover, Table 35 through Table 37 show that the average difference between FE model results and dynamic testing is 9.29% at 10 mph, 8.81% at 20 mph and 9.44% at 25 mph, respectively. For this bridge, the differences between FE model and dynamic testing are mainly due to the reason of dynamic impact which was not taken into consideration in the FE model. According to the dynamic impact analysis, the dynamic impact varied from 10% to 20% at speed range of 10 mph to 25mph. Therefore the coefficient of variation for the comparison with dynamic test is reasonable. Overall, the FE model exhibited good agreement with the field test data under railcar loading and can be used for further load rating analysis and future evaluation of bridge performance.

Table 34 - Comparison of results from FE model and field-testing for the HX Draw Bridge (Static test)

Comparison	FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2568	110	110	0	0.00%
B2564	110	111	1	-0.90%
B2567	110	105	5	4.76%
B2562	110	105	5	4.76%
B2563	150	150	0	0.00%
B2569	148	140	8	5.71%
B2566	110	105	5	4.76%
B2574	110	109	1	0.92%
		Average	3.13	2.72%

Table 35 - Comparison of results for the HX Draw Bridge at a train speed of 10 mph.

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2568	Peak	198	200	2	-1.00%
B2564	Peak	198	220	22	-10.00%
B2567	Peak	198	170	28	16.47%
B2562	Peak	198	225	27	-12.00%
B2563	Peak	175	177	2	-1.13%
B2569	Peak	175	180	5	-2.78%
B2566	Peak	123	150	27	-18.00%
B2574	Peak	123	147	24	-16.33%
			Average	16.50	9.71%

Table 36 - Comparison of results for the HX Draw Bridge at a train speed of 20 mph

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2568	Peak	198	200	2	-1.00%
B2564	Peak	198	223	25	-11.21%
B2567	Peak	198	188	10	5.32%
B2562	Peak	198	225	27	-12.00%
B2563	Peak	175	177	2	-1.13%
B2569	Peak	175	190	15	-7.89%
B2566	Peak	123	150	27	-18.00%
B2574	Peak	123	143	20	-13.99%
			Average	16.00	8.81%

Table 37 - Comparison of results for the HX Draw Bridge at a train speed of 25 mph

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2568	Peak	198	203	5	-2.46%
B2564	Peak	198	225	27	-12.00%
B2567	Peak	198	188	10	5.32%
B2562	Peak	198	230	32	-13.91%
B2563	Peak	175	185	10	-5.41%
B2569	Peak	175	182	7	-3.85%
B2566	Peak	123	150	27	-18.00%
B2574	Peak	123	144	21	-14.58%
			Average	17.38	9.44%

Verification of the Raritan Valley Line MP 31.15(Middle Brook) Bridge Model

Figure 70 and Figure 71 show the comparison between the FE model and field-testing data for the Raritan Valley Line MP 31.15 Bridge. The section modulus for the FE model is modified to obtain good agreement with the testing data (please refer to previous section in testing part). Table 38 shows the differences as percentages between results from the FE model and field-testing data. Additionally, Table 38 shows that the average difference between FE model and field-testing data is 2.06% that is below 5%. This means that the results from FE model show excellent correlation with the testing data.

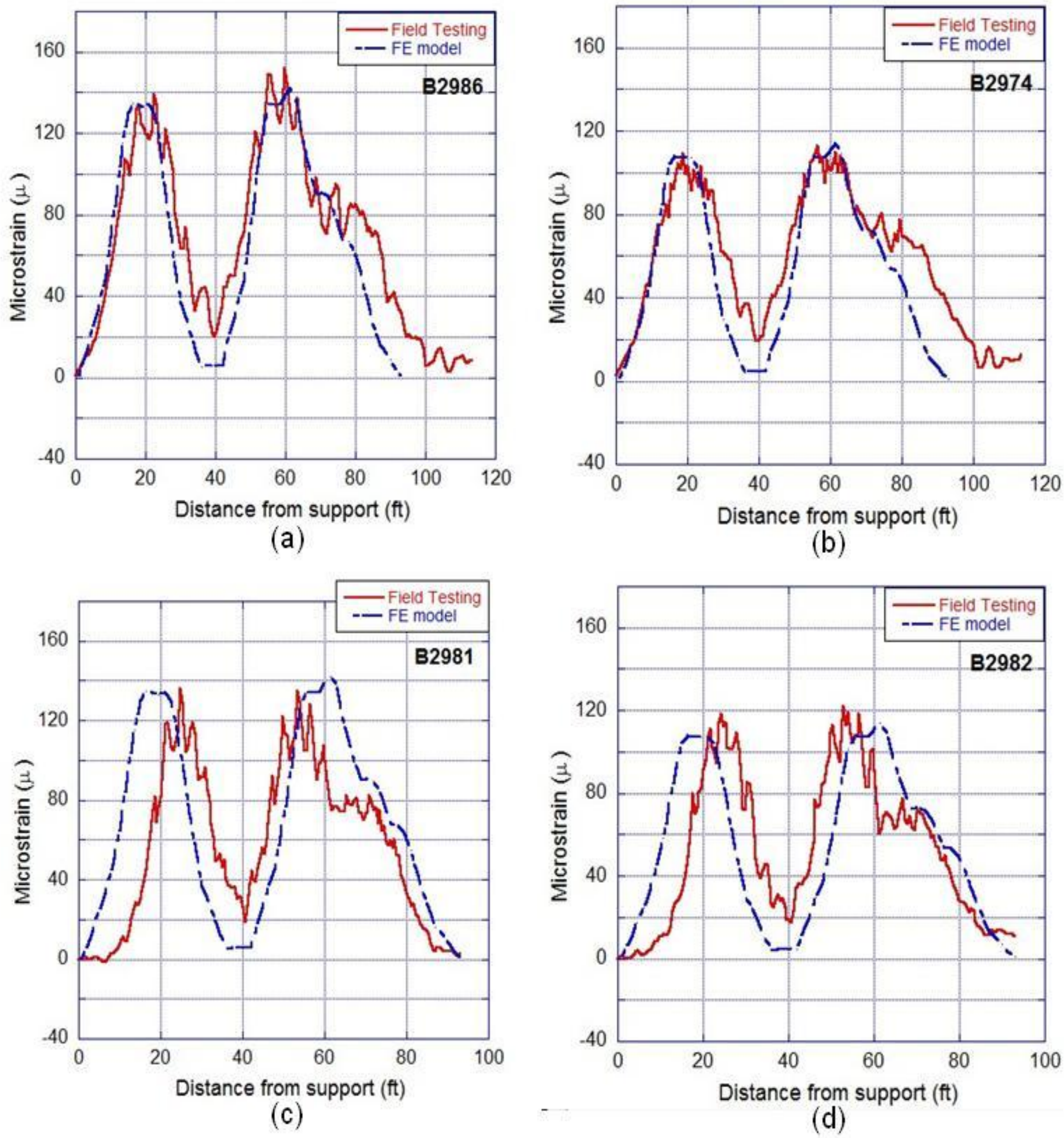


Figure 70. Comparison of strain data collected in midspan data with FE model: *(a)* B2986 Run 11, *(b)*B2974 Run 11,*(c)*B2981 Run 9, and*(d)*B2982 Run 9 for the Raritan Valley Line MP 31.15 (Middle Brook) Bridge

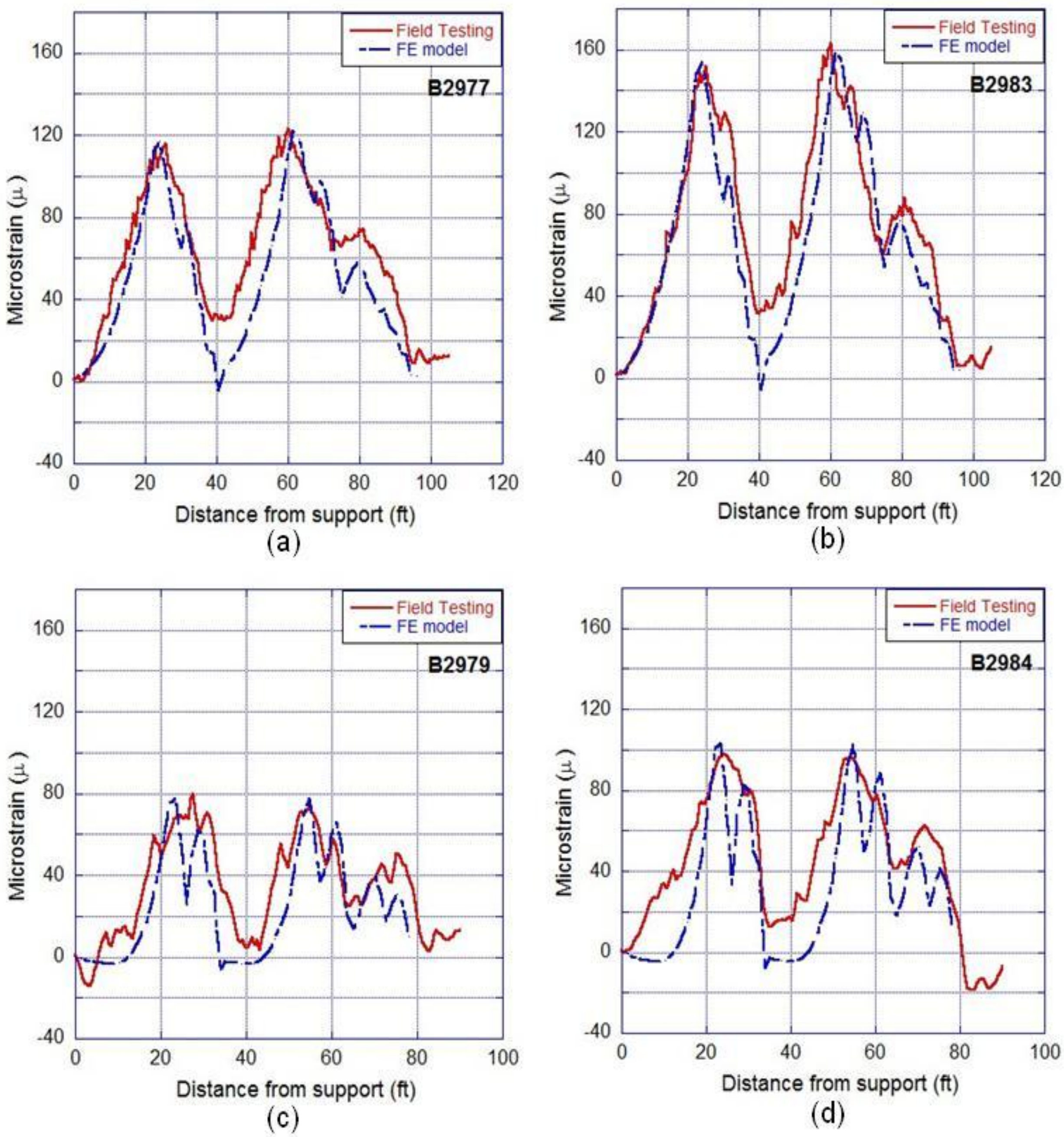


Figure 71. Comparison of strain data collected in cutoff point data with FE model: *(a)* B2977 Run 11, *(b)* B2983 Run 11, *(c)* B2979 Run 11, and *(d)* B2984 Run 11 for the Raritan Valley Line MP 31.15 (Middle Brook) Bridge

Table 38 - Comparison of results from FE model and field-testing for the Raritan Valley Line MP 31.15 Bridge

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B2986	Peak	140	150	10	-6.67%
B2974	Peak	110	110	0	0.00%
B2981	Peak	140	138	2	1.45%
B2982	Peak	110	120	10	-8.33%
B2977	Peak	120	120	0	0.00%
B2983	Peak	160	160	0	0.00%
B2979	Peak	80	80	0	0.00%
B2984	Peak	100	100	0	0.00%
			Average	2.75	2.06%

Verification of the North Jersey Coast Line 0.39 (River Draw) Bridge Model

Figure 72 shows the comparison between the experimental data and analysis results using a calibrated model. The section properties, various boundary conditions, and various connection types were used to calibrate the FE model developed using ABAQUS software. Table 39 shows the difference between FE model and field-testing in terms of percentage.

Table 39 shows that the average difference between FE model and field-testing is 7.15% that is below 10%. The differences can be attributed to the dynamic impact during the test. The above discussion suggests that the FE model results are in good agreement with those from the field tests.

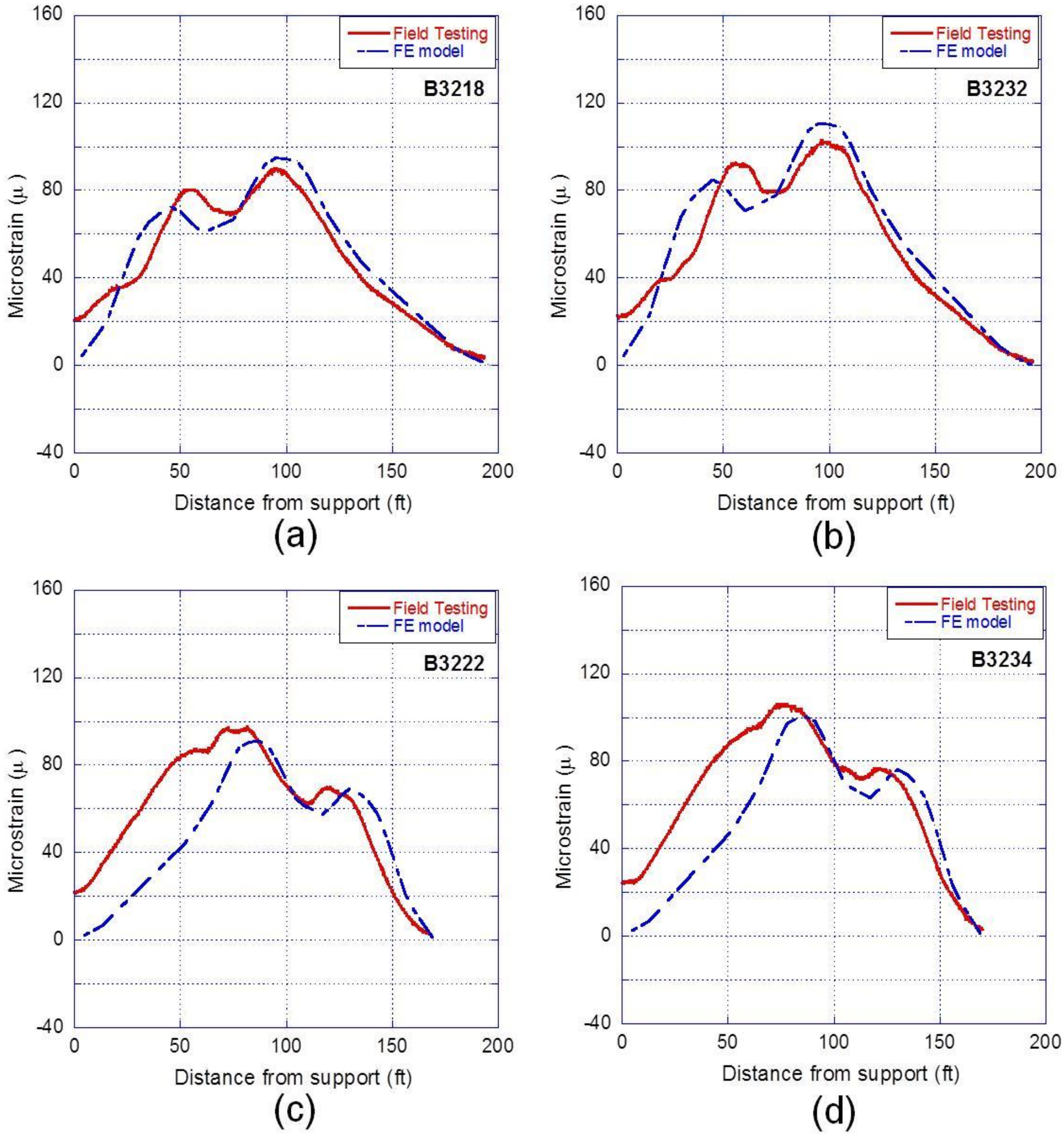


Figure 72. Comparison of strain data at cutoff locations of girders: *(a)*B3218(Test Run#1), *(b)*B3232(Test Run#1), *(c)* B3222 (Test Run#1), and *(d)* B3234 (Test Run#1) for the North Jersey Coast Line 0.39 (River Draw) Bridge

Table 39 - Comparison of results from FE model and field-testing for the North Jersey Coast Line 0.39 Bridge

Comparison		FE value (με)	Testing value (με)	Difference (με)	Percentage Difference
B3218	Peak	95	90	5	5.56%
B3232	Peak	110	100	10	10.00%
B3222	Peak	90	98	8	-8.16%
B3234	Peak	100	105	5	-4.76%
			Average	7.00	7.15%

Dynamic Factor Investigation for the Bergen County Line MP 5.48 (HX Draw) Bridge

In this section, the dynamic factor of a 60 ft approach span was investigated based on the static and dynamic tests data. Table 40 shows the dynamic factor at various speeds and at different locations. It is observed that the impact factor increases with an increase in train speed increases. The maximum dynamic factor calculated based on field testing data is 1.25, while the dynamic factors calculated using AREMA provisions is 1.4. Therefore, the calculation of impact using AREMA provisions is slightly more conservative. That is mainly because the AREMA calculation considers the worst scenarios for different situations which could arise during the lifetime of the bridge.

<u>Two-dimensional (2D) train-bridge dynamic model</u>

To study the dynamic responses of railroad bridges under moving train, a two-dimensional (2D) train-bridge dynamic model is developed, as shown in Figure 73. Figure 73 shows a typical train composed of several identical vehicles running over a railroad bridge. For each vehicle, it is composed of one car body, two identical bogies, four identical wheel sets, and the primary and secondary suspension systems are modeled as linear spring-dashpot units.[(28)]

In this part of the study, it is assumed that the wheel sets of each vehicle are kept in full contact with the bridge at all times and the separation between the wheel sets and bridge is not allowed, so the dynamic responses of the bridge and vehicle are linearly coupled, which can be computed using conventional time integration methods without iterations. The carbody and two bogies are each assigned two DOFs, which are vertical displacement and rotation about the center point. The equations of motions for vehicle components can be described entirely using second-order ordinary differential equations in the time domain, which are solved by applying the D’Alembert’s principle.

Table 40 - Summary of the impact factor for the Bergen County Line MP5.48 Bridge

Location		Dynamic Strain (με) 10 mph	Dynamic Strain (με) 20 mph	Dynamic Strain (με) 25 mph	Static value (με)	Dynamic Factor (1) 10 mph	Dynamic Factor (1) 20 mph	Dynamic Factor (1) 25 mph	Dynamic Factor Calculated Based on AREMA (2)
Span 3 Track 2 north girder	Midspan	203.0	201.1	203.7	196.6	1.03	1.02	1.04	10 mph
	Second cutoff	180.0	180.6	194.0	169.3	1.06	1.07	1.15	1.27
	First cutoff	139.3	140.9	141.2	117.2	1.19	1.2	1.21	20 mph
Span 3 Track 2 south girder	Midspan	212.6	220.8	229.4	188.1	1.13	1.17	1.22	1.37
	Second cutoff	188.0	192.0	192.1	169.3	1.11	1.13	1.13	25 mph
	First cutoff	148.5	148.8	151.9	122.0	1.22	1.22	1.25	1.41
Span 3 Track 1 north girder	Midspan	223.0	228.0	231.0	188.3	1.18	1.21	1.23	Design Value Without Speed Reduction Factor 1.54

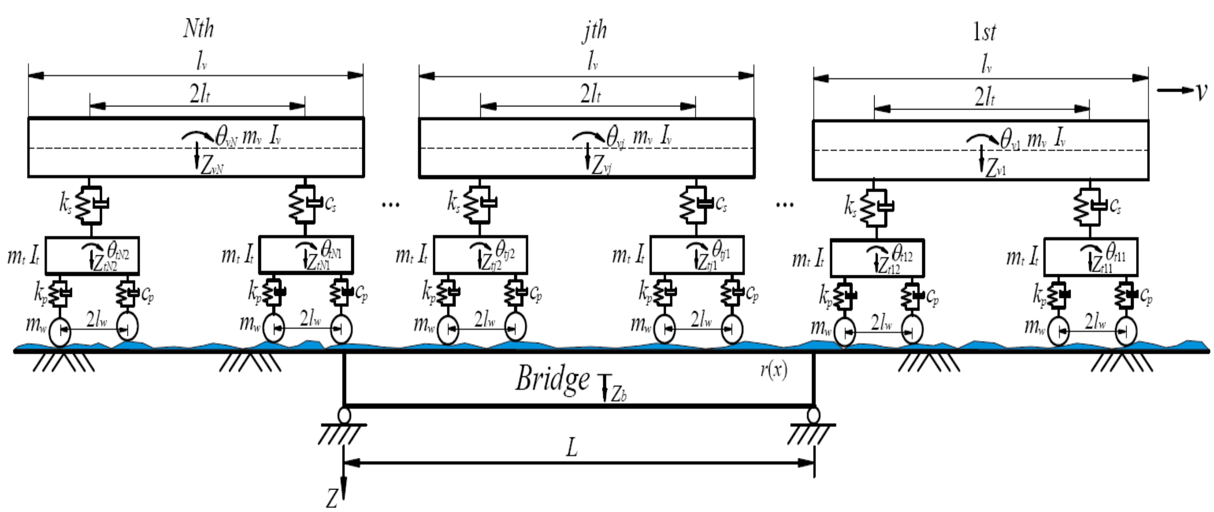


Figure 73. Two-Dimensional (2D) Train-bridge dynamic interaction model

The simple bridge is modeled as a linear elastic Bernoulli-Euler beam with identical sections. Using modal superposition method, the equation of motion for the bridge subjected to moving vehicles can be written as a series of second-order ordinary differential equations with generalized displacements.

By combining these two parts of differential equations, the equations of motion including the simple bridge and all of the vehicles in the modal space can be presented in a matrix form as Equation (10):[(29)]

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{F\} \tag{10}$$

where [M], [C], [K] denote the mass, damping and stiffness matrices; $\ddot{U}$, $\dot{U}$, and U are the vectors of displacement, velocity, and acceleration, respectively; and {F} represents the vector of exciting forces applied to the dynamic system. To compute both the dynamic responses of the simple bridge and moving vehicles, the generalized matrix equation of motion given in Equation (10) will be solved using a step-by-step integration method, i.e., the Newmark method. In this study, β=1/4 and γ=1/2 are selected, which implies a constant acceleration with unconditional numerical stability.[(30)]

The 2-D model can only consider the vertical effect of dynamic impact without the rocking effect. The dynamic impact (vertical effect) for different cars is listed in Figure 74. From the analysis results, the maximum impact factor within 120 mph is 7% for passenger train, 6% for 286K freight car and 4% for 263K freight car. The passenger train has the highest impact factor within 120 mph followed by 286-kips freight car, while the 263-kips freight car has the lowest impact factor.

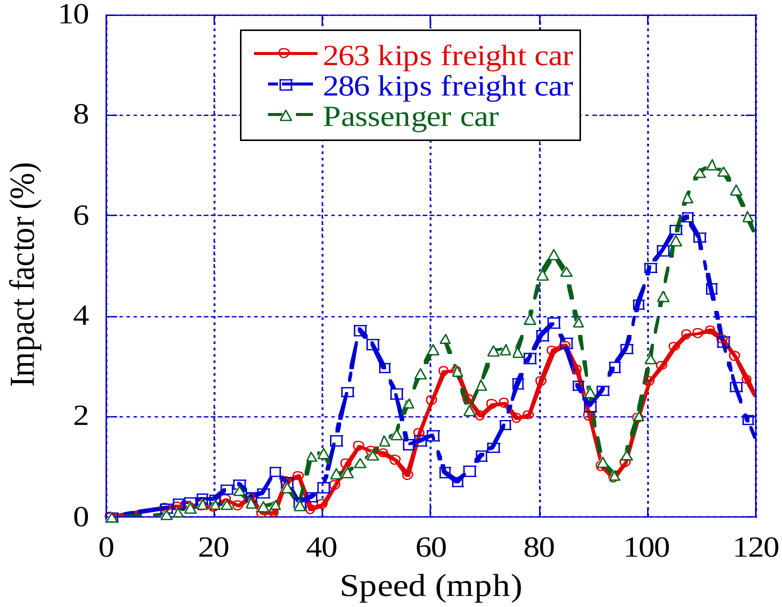


Figure 74. Impact factor for different cars

Fatigue Rating Based on AREMA Specification

Conway discussed the practical application of the rating rules using AREMA specifications and stated that the fatigue rating is just used to indicate the remaining service life of the structure qualitatively.[(31)] For example, if the actual loading is less than the fatigue rating, it means that the fatigue damage does not accumulate and the structure may still have a relatively long remaining fatigue life. Even if the actual loading is considerably higher than the fatigue rating, the structure may still have a moderately long remaining service life.

In the Inspection Report Cycle 4 of Raritan Valley Line MP 31.15, it was noted that the fatigue rating is based on the allowable fatigue stress range specified by AREMA and that fatigue life evaluation instead of fatigue rating would provide a better indication of fatigue performance.[(22)] In this section, both the fatigue rating, as well as the evaluation of fatigue remaining life were performed for all five bridges.

Fatigue Rating

As an illustrative example, the fatigue rating of Span 3, Girder 1, at 8.5 ft from the support of the HX Drawbridge Bridge is presented herein. The fatigue ratings of all five bridges are summarized in Table 41.

(1) Choose the number of stress cycles from AREMA 2010, Chapter 15, Page 15-1-23, Table 15-1-7.

For a through girder with a span length of 60 ft, the number of stress cycles is > 2×10^6

(1) Choose fatigue stress category from AREMA 2010, Chapter 15, Page 15-1-23, Table 15-1-9.

Choose Category D for riveted connections

(2) Select allowable fatigue stress range (S_{Rfat}) from AREMA 2010, Chapter 15, Page 15-1-23, Table 15-1-16.

The allowable fatigue stress range is 5 ksi

(3) Calculate net section modulus

The net section modulus is 1,589 in^3 for as-inspected section at midspan

(4) Calculate live load moment includes impact effect (M_{LL}+I)

The live load moment due to E80 rail car plus impact is 2241.6 k-ft

(5) Calculate stress range (S_R) resulting from loading

The stress range resulting from loading is $(M_{LL}+I)/S$, where S is the section modulus. The calculated stress range is 16.92 ksi

(6) Final step, the fatigue rating is calculated as

$$S_{Rfat}/S_R*80= 25.1$$

Therefore, the fatigue rating of the typical 60 ft girder at midspan using as-inspected section properties is E25, which means that the maximum allowable weight of rail car is E25 in order to fulfill the requirements of allowable fatigue stress range and number of stress cycles as specified in AREMA.

Similar steps were followed to obtain the results of the fatigue rating for the remaining four bridges that are summarized in Table 41.

Table 41 - Fatigue rating for the five selected bridges

Line	MP	Name	Member	Location	Fatigue Rating
Main Line	15.95	Broadway	G2, Span 2	8.6'	E235
Main Line	15.14	Straight St.	FB 20, Span 1, Bay 4	Midspan	E55
Bergen County	5.48	HX Draw	Span 3 G1, Track 2	8.5'	E23
Raritan Valley	31.15	Middle Brook	G8, Span 3	Midspan	E21
North Jersey Coast	0.39	River Draw	88' Girder	24.5'	E22

Fatigue Life Evaluation

In addition to the fatigue rating, the Commentary of Chapter 15 in the AREMA specification in Part 9, specifies that if the actual stress cycles could be estimated from the operation records or survey, then the total number of stress cycles can be evaluated based on the effective stress range and the stress-cycle curve (S-N) curve. The effective stress range can be calculated using the following equation:

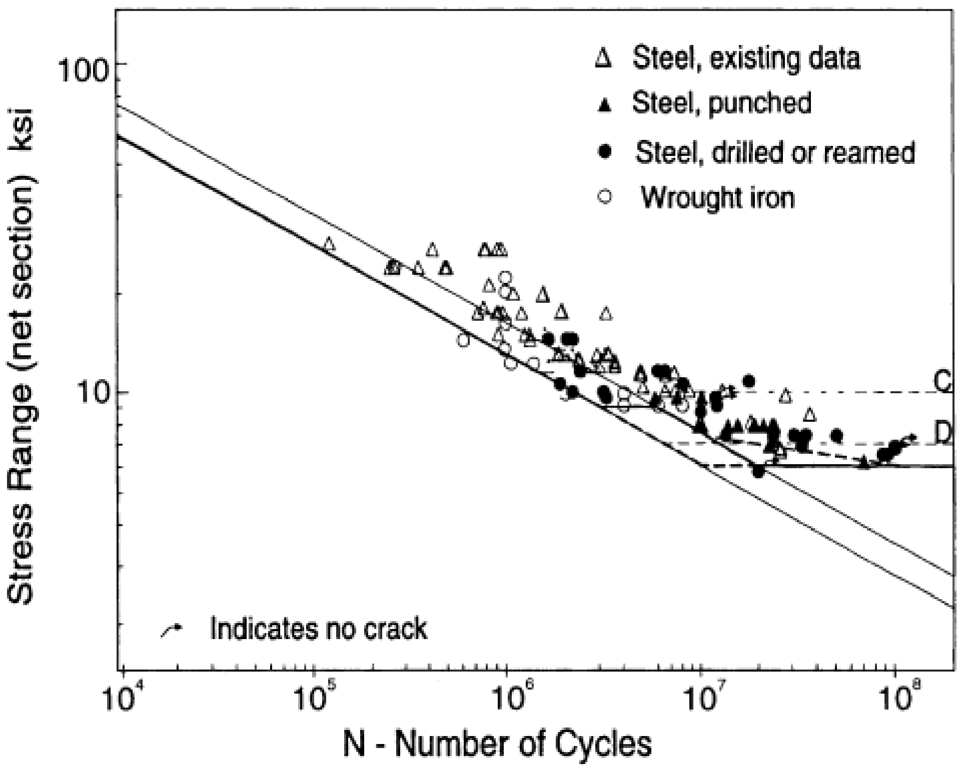
$$S_{Re} = \alpha(\sum \gamma_i S_{Ri}^3)^{1/3} \tag{11}$$

where α is a reduction ratio, S_{Ri} is the stress range corresponding to number of occurrences of n_i, and γi is the ratio of n_i to the total number of cycles.

AREMA also specifies that the combination of S_{Re} and number of cycles (Nv) should be less than the S-N curves shown in Figure 75 and Figure 76.

Based on the information specified in the commentary of the AREMA specification, the fatigue life evaluation was performed for all five bridges.

Depending on the number of daily trips and the load level, the addition of a 286-kips freight railcar will have a significant effect on the fatigue life of the structure. Therefore, in addition to Normal and Maximum load rating, there is a need to evaluate the effect of adding the 286 kips railcar on the fatigue life of the bridge.



For riveted bridge components For design only:	For optional evaluation of drilled or reamed bridge components
$N=2.183 \times 10^9\ S_r^{-3}\ S_r > 9$ ksi For evaluation: $N=2.183 \times 10^9\ S_r^{-3}\ S_r > 9$ ksi $N=4.446 \times 10^9\ S_r^{-3}\ 9\text{ ksi} > S_r > 6$ ksi Fatigue limit: $(S_r)_{fl} = 6$ksi	(see 9.7.3.4.2 Fatigue) $N=2.183 \times 10^9\ S_r^{-3}\ S_r > 9$ ksi $N=4.446 \times 10^9\ S_r^{-3}\ 9\text{ ksi} > S_r > 7.65$ ksi $N=2.465 \times 10^{15}\ S_r^{-9.5}\ 7.65\text{ ksi} > S_r > 6$ ksi Fatigue limit: $(S_r)_{fl} = 6$ ksi

Figure 75. S-N curve for riveted bridge components (Figure 15-9-4, AREMA)[9]

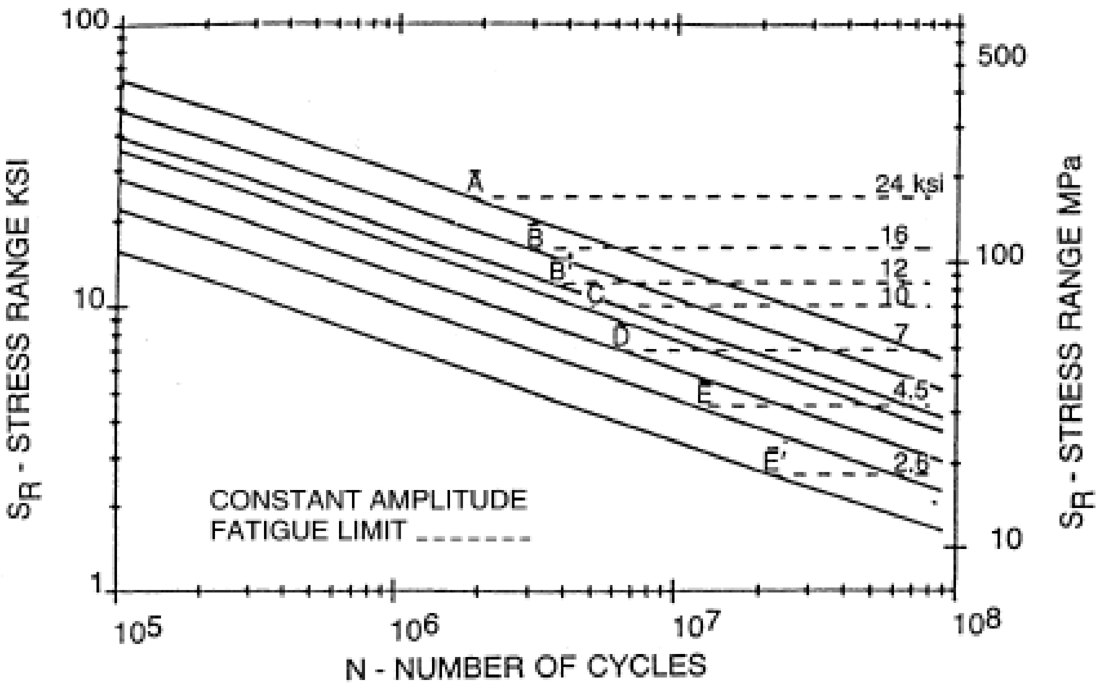


Figure 76. S-N curves for fatigue evaluation (Figure 15-9-5, AREMA)[9]

Due to absence of actual train traffic information over years, the following approach with various assumptions was made to proceed with the calculations and simplify the analysis. These assumptions were verified by NJ Transit to help achieving preliminary results for fatigue life estimation.

Fatigue Analysis Approach:

Assumption a. What would be the Passenger Train Schedule during the bridge's second 50 years' service life:

According to NJ Transit, the current volume of train traffic consists of revenue trains and non-revenue trains, freight and other NJ Transit lines that use the bridges. Based on the revenue passenger train schedule and the observations made during the field testing, it is found that the passenger train travels over the bridge 24 to 36 times daily with one locomotive and 6 to 10 passenger railcars (Actual volumes depend on the bridge location and days (weekday or weekends)). Therefore, it is assumed that the passenger train travels over the bridge 30 times with one locomotive and 10 passenger rail cars. It is assumed that the service life of a typical bridge is 100 years. It is noted that these train counts above excluded the counts from non-revenue trains.

Assumption b. What is the expected volume of 286-kips railcars and their schedule:

Data obtained from NJDOT about the current or expected use of 286 railcars on HX Draw Bridge shows that 7000 286-kips rail car will travel over HX Draw Bridge per year, while 8800 for River Draw and 8000 for the other three bridges.

Assumption c. What would be the Passenger Train Schedule during the bridge's first 50 years' service life:

(c1) The volume of passenger train traffic on the bridge during its first 50 years of service was the same as current volume.
(c2) The volume of passenger train traffic on the bridge during its first 50 years of service was 1.2 times as current volume.
(c3) The volume of passenger train traffic on the bridge during its first 50 years of service was 1.5 times as current volume.

Table 42 through Table 45 summarizes the analysis results for different assumptions. It is observed that remaining life of most of the bridges will reduced by 35% to 51% for assumption c1, 39% to 94% for assumption c2, and 42% to 100% for assumption c3. Therefore, the utilization of 286 kips railcar, at a minimum, will reduce the remaining service life of the bridges by a percentage of 35-50%. The exact impact of utilizing 286 kips railcar on service life of bridges can be evaluated after receiving more accurate information on the train traffic volumes from NJ Transit. Please note that the remaining life was estimated based on the stress-cycle curve (S-N curve) specified in Figure 15-9-4 of AREMA specification and that the actual service life might differ from the estimation based on AREMA specification.

Table 42 – Fatigue life evaluation of selected bridges using Assumptions a, b, and c1

Bridge	Member	Location	Remaining Life (years)		% Reduction of Remaining Life
			W/O 286	With 286	
Main Line MP 15.95	G2, Span 2	8.6'	> 50	> 50	-
Main Line MP 15.14	FB 20, Span 1, Bay 4	Midspan	46	25	46%
Bergen County MP 5.48	Span 3 G1, Track 2	8.5'	74	48	35%
Raritan Valley MP 31.15	G8, Span 3	Midspan	43	21	51%
North Jersey Coast MP 0.39	88' Girder	68	68	44	35%

Table 43 – Fatigue life evaluation of selected bridges based Assumptions, a, b, and c2

Bridge	Member	Location	Remaining Life (years)		% Reduction of Remaining Life
			W/O 286	With 286	
Main Line MP 15.95	G2, Span 2	8.6'	> 50	> 50	-
Main Line MP 15.14	FB 20, Span 1, Bay 4	Midspan	36	15	58%
Bergen County MP 5.48	Span 3 G1, Track 2	8.5'	64	39	39%
Raritan Valley MP 31.15	G8, Span 3	Midspan	17	1	94%
North Jersey Coast MP 0.39	88' Girder	68	33	12	64%

Table 44 - Fatigue life evaluation of selected bridges based on Assumptions a, b, and c3

Bridge	Member	Location	Remaining Life (years)		% Reduction of Remaining Life
			W/O 286	With 286	
Main Line MP 15.95	G2, Span 2	8.6'	> 50	> 50	-
Main Line MP 15.14	FB 20, Span 1, Bay 4	Midspan	22	2	91%
Bergen County MP 5.48	Span 3 G1, Track 2	8.5'	59	34	42%
Raritan Valley MP 31.15	G8, Span 3	Midspan	2	0	100%
North Jersey Coast MP 0.39	88' Girder	68	19	0	100%

Table 45 – Comparison of Various Assumptions and the Reduction of Fatigue Life

Bridge	Member	Location	% Reduction of Remaining Life		
			c1	c2	c3
Main Line MP 15.95	G2, Span 2	8.6'	-	-	-
Main Line MP 15.14	FB 20, Span 1, Bay 4	Midspan	46%	58%	91%
Bergen County MP 5.48	Span 3 G1, Track 2	8.5'	35%	39%	42%
Raritan Valley MP 31.15	G8, Span 3	Midspan	51%	94%	100%
North Jersey Coast MP 0.39	88' Girder	68	35%	64%	100%

RESULTS AND COMPARISON

Main Line MP 15.95 Bridge

Table 46 shows the loading rating results for the Main Line MP 15.95 Bridge. Based on the simple beam analysis, the maximum demand over capacity (D/C) ratio is 111%, which means certain repairs are needed to improve the performance of the bridge to accommodate 286 kips railcar loading.

Although the FE analysis shows higher load capacity, given the uncertainty in estimating the effect of the 286-kips on the remaining fatigue life, the RT will conservatively follow the AREMA requirements in making their recommendations, with a minimum level for safety of providing a D/C of 80%. Data from field tests were used to calibrate the FE model to help provide more accurate results. This model calibration was implemented in changes of the boundary conditions to be fix-pin rather than simply supported as was assumed in the simple beam analysis. The differences between FE model and simple-beam analysis may come from the boundary condition, member connectivity and load distribution. The final recommendations took into account the variation of the load rating results between the three different approaches: 1) AREMA's simple beam analysis, 2) Finite Element Analysis and 3) Field Testing data. For the Equivalent Cooper E load for 286-kips rail car, the difference between the FE model and simple beam analysis (e.g. at section 8.65' from support, the equivalent load is E65 from FE model and E59 from simple beam analysis) can be attributed to the different boundary condition in the simple beam analysis and FE model analysis. Please note that the calculation of Equivalent Cooper E load for 286-kips rail car based on FE model is similar to simple beam analysis (the moments for calculations are from FE model).

Table 46 - Load rating results for the Main Line MP 15.95

As Inspected		**Equivalent Cooper E Load for 286-kips Rail Car**		**Cooper E Rating**		**Comparison**	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.65' from support*	E65	E59	375	53	17%	111%
	11' from support	E64	E59	247	58	26%	102%
	14.4' from support	E62	E60	211	62	29%	97%
	Midspan	E62	E60	199	65	31%	92%
Maximum load rating	8.65' from support*	E65	E59	552	85	12%	69%
	11' from support	E64	E59	367	93	17%	63%
	14.4' from support	E62	E60	316	99	20%	61%
	Midspan	E62	E60	297	103	21%	58%

Main Line MP 15.14 Bridge

As shown in Table 47, for Main Line MP 15.14, the critical member is Floor Beam 20. The Cooper E rating is 29 at normal level and 47 at maximum level according to the FE model, while the equivalent Cooper E rating is E67 both at normal and maximum, which means 286-kips rail-car loading is 231 percent of normal carrying capacity and 143 percent of maximum carrying capacity. From the simple beam analysis, it also reflects the 286-kips rail-car load exceeds the capacity, 191 percent, at the normal level and 120 percent at the maximum level. However, as per NJ Transit Engineers, Floor Beam 20 is under an abandoned track and does not reflect the condition of the structure that is supporting the active tracks. Therefore, the critical structural member under the active tracks was also investigated. Based on the simple beam analysis, the maximum demand over capacity ratio is 113%, which means certain repair is needed to improve the performance of the bridge. However, the FE analysis shows higher load capacity due to the fact that the boundary condition is different with what is assumed in the simple beam analysis. Based on field-testing data, the boundary condition should be pin-pin rather than simply supported, which is assumed in the simple beam analysis. This will lead to the high load-carrying capacity of girders, but the boundary condition will not affect the load-rating results of the floorbeam. The maximum demand over capacity is 52% using FE model analysis. For the Equivalent Cooper E load for 286-kips rail car, the difference between FE model and simple beam analysis (e.g. at section 14.5’ from support of G37, equivalent load is E43 from FE model analysis and E59 from simple beam analysis) can be attributed to the boundary condition as well as the large skew of this bridge.

Table 47 - Rating results for Main Line MP 15.14 using as-inspected section properties

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	Midspan of FB20	E67	E67	29	35	231%	191%
	G37 10.6' from support	E44	E58	139	62	32%	94%
	G37 14.5' from support	E43	E59	108	52	40%	113%
	G37 19.6' from support	E42	E57	99	53	42%	108%
	G37 Midspan	E40	E54	77	57	52%	95%
	G28 section 2	E62	E52	175	54	35%	96%
	G28 section 5	E62	E52	188	56	33%	93%
	G28 section 7	E53	E49	124	61	43%	80%
	G28 Midspan	E47	E47	143	64	33%	73%
	G29 Midspan	E47	E56	229	78	21%	72%
Max. load rating	Midspan of FB20	E67	E67	47	56	143%	120%
	G37 10.6' from support	E44	E58	179	104	25%	56%
	G37 14.5' from support	E43	E59	183	92	23%	64%
	G37 19.6' from support	E42	E57	168	94	25%	61%
	G37 Midspan	E40	E54	129	97	31%	56%
	G28 section 2	E62	E52	282	87	22%	60%
	G28 section 5	E62	E52	303	91	20%	57%
	G28 section 7	E53	E49	199	97	27%	51%
	G28 Midspan	E47	E47	229	102	21%	46%
	G29 Midspan	E47	E56	348	119	14%	47%

Bergen County MP 5.48 (HX Draw) Bridge

As shown in Table 48 through Table 53, for Bergen County Line MP 5.48 Bridge, the critical load rating (midspan of south girder under Track 1 in Span 3) for Cooper E load is 43 at the normal level and 65 at the maximum level, while the equivalent Cooper E load for a 286-kips Rail car is E57 at the normal level, according to the FE model. Therefore, 286-kips rail car loading will exceed normal rating capacity. In terms of percent capacity, 286-kips rail-car loading is 132 percent of normal carrying capacity and 87 percent of maximum carrying capacity. Based on field testing and the FE model, there are a total of six sections where demand over capacity ratio exceeds 100 percent. In addition, the most critical member is the midspan of south girder underneath Track 1 in Span 3. This is different from the Inspection Report Cycle 4, which did not consider the upgrade of the bridge after the fourth inspection.[21] The section properties used in the FE model are based on the information provided in the latest inspection report, with the updates from NJ Transit regarding the recent repair. As a result of recent repair, the compression flange at section 3 also controlled the rating that is showed in Table 53. The maximum demand over capacity ratio is 105 percent which certain repair is needed to improve the load carrying capacity of the girders.

Table 48 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 2 in Span 3) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.5' from support	E60	E60	63	64	95%	94%
	Midspan*	E57	E56	53	54	108%	104%
Maximum load rating	8.5' from support	E60	E60	94	97	64%	62%
	Midspan*	E57	E56	80	82	71%	68%

*Controls

Table 49 - Rating results for Bergen County MP 5.48 using as-inspected section properties (south girder under Track 2 in Span 3) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.5' from support	E60	E60	66	64	91%	94%
	Midspan*	E57	E56	51	54	112%	104%
Maximum load rating	8.5' from support	E60	E60	99	97	61%	62%
	Midspan*	E57	E56	76	82	75%	68%

*Controls

Table 50 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 1 in Span 3) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	Midspan	E57	E56	52	54	109%	104%
Maximum load rating	Midspan	E57	E56	78	82	73%	68%

Table 51 - Rating results for Bergen County MP 5.48 using as-inspected section properties (south girder under Track 1 in Span 3) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	Midspan	E57	E56	43	44	132%	127%
Maximum load rating	Midspan	E57	E56	65	67	87%	84%

Table 52 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 2 in Span 2) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.5' from support	E60	E60	54	53	111%	113%
	Midspan*	E57	E56	44	45	130%	124%
Maximum load rating	8.5' from support	E60	E60	80	80	75%	75%
	Midspan*	E57	E56	66	68	86%	82%

*Controls

Table 53 - Rating results for Bergen County MP 5.48 using as-inspected section properties (compression flange at section 3) (Current)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.5' from support	E60	E60	57	59	105%	102%
Maximum load rating	8.5' from support	E60	E60	87	90	69%	67%

Raritan Valley MP 31.15 (Middle Brook) Bridge

As shown in Table 54, for Raritan Valley Line MP 31.15, the load rating for the midspan of G8 (critical member) is 63 at normal level and 95 at maximum level. The Equivalent Cooper E load for 286-kips rail car is E62. In terms of percent capacity, 286-kips rail car is 98 percent of normal carrying capacity and 65 percent of maximum carrying capacity. The differences between the results of the FE model and the simple beam analysis are a result of the different section properties used during the analysis. Based on the calibrated FE model, the member has minor section losses while the simple beam analysis considers the section loss more conservatively based on Inspection Report Cycle 4.[22]

Table 54 - Rating results for Raritan Valley MP 31.15 using as-inspected section properties

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	Midspan of G8	E62	E61	63	59	98%	103%
Maximum load rating	Midspan of G8	E62	E61	95	89	65%	69%

North Jersey Coast Line MP 0.39 (River Draw) Bridge

As shown in Table 55, for the North Jersey Coast Line MP 0.39, the load rating of the critical member is 62 at a normal level and 100 at a maximum level. The equivalent Cooper E rating for a 286-kips rail car is E60. In terms of percent capacity, a 286-kips rail car has 97 percent of normal carrying capacity and 60 percent of maximum carrying capacity. The differences of results between the FE model and simple beam analysis could also be addressed by the difference of section properties as explained in the Raritan Valley MP 31.15 section.

Table 55 - Rating results for North Jersey Coast Line MP 0.39 using as-inspected section properties

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	Midspan	E47	E47	52	62	90%	76%
	24.5' from support*	E60	E61	62	47	97%	130%
Maximum load rating	Midspan	E47	E47	82	99	57%	47%
	24.5' from support*	E60	E61	100	70	60%	87%

Repair Recommendations and Cost Analysis Based On Load Rating Using FE Model

Based on the load rating result using FE model, the following section A to G need to be repaired to fulfill the demand over capacity ratio of 100% while section A to J need to be repaired to fulfill the demand over capacity ratio of 80%:

- Section A: Midspan of north girder underneath Track 2 in Span3 of Bergen County Line MP 5.48 (HX Drawbridge);
- Section B: Midspan of south girder underneath Track 2 in Span 3 of Bergen County Line MP5.48 (HX Drawbridge;)
- Section C: Midspan of north girder underneath Track 1 in Span 3 of Bergen County Line MP 5.48 (HX Drawbridge);
- Section D: Midspan of south girder underneath Track 1 in Span 3 of Bergen County Line MP 5.48 (HX Drawbridge);
- Section E: Midspan of north girder underneath Track 2 in Span 2 of Bergen County Line MP 5.48 (HX Drawbridge);

- Section F: 8.5’ from west support of north girder underneath Track 2 in Span 2 of Bergen County Line MP 5.48 (HX Drawbridge); and
- Section G: FB20 (Span 1, Bay 4) of ML MP 15.14.
- Section H: Raritan Valley Line, Midspan, 40’ girder
- Section I: River Draw, 24.5' from support, 88' girder
- Section J: River Draw, Midspan, 88' girder

Repair Recommendation without Speed Restriction

Based on the load rating using FE analysis, the RT recommends the following improvements as shown in Table 56 or Table 58 if speed restrictions will not be utilized. Please note that the recommendations of Table 56 are made theoretically to satisfy a demand-over-capacity ratio of less than 100 percent at a normal rating level, while in Table 58 recommendations are made to satisfy a demand-over-capacity percentages of below 80 percent at a normal rating level. Further engineering review is needed to verify the feasibility of the recommendations.

Table 56 - Recommended section modulus (demand over capacity less than 100%)

Location	Current S(bot)* in^3	Recommended S(bot) in^3	Recommendation
HX, Midspan, NG, T2, S3	2,350.0	2,506.7	Add 1/4 in. thickness cover plate to the bottom
HX, Midspan, SG, T2, S3	2,250.0	2,506.7	Add 3/8 in. thickness cover plate to the bottom
HX, Midspan, NG, T1, S3	2,334.0	2,506.7	Add 1/4 in. thickness cover plate to the bottom
HX, Midspan, SG, T1, S3	1,938.0	2,506.7	Add 7/8 in. thickness cover plate to the bottom
HX, Midspan, NG, T2, S2	1,977.0	2,506.7	Add 7/8 in. thickness cover plate to the bottom
HX, 8.5’ from support NG, T2, S2	1,210.0	1,320	Add 1/4 in. thickness cover plate to the bottom
ML 15.14, FB20	64.2	**	Since long holes occur at web of FB 20 and FB 21 near the connection with G5, and 100% loss of end section of FB 19 at connection with cross girder E4-5 (Polytran Engineering Associates, P.C., 2007), it is recommended that FBs 19, 20, and 21 be replaced.

*S(bot) = Section properties based on validated FE model.

** No recommended S(bot) since it is recommended to replace FB20

As a result of repair recommendation, the increase in the thickness of bottom flange will led to the increase of the section modulus of the top side. The compression load rating will changed as following in Table 57. The maximum demand over capacity ratio is 103%, which is close to 100%.

Table 57 - Rating results for Bergen County MP 5.48 using as-inspected section properties (compression flange at section 3) (after repair)

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car		Cooper E Rating		Comparison	
Rating Type	Location	FE Model (1)	Simple Beam Analysis (2)	FE Model (3)	Simple Beam Analysis (4)	Demand over Capacity Ratio (1)/(3)	Demand over Capacity Ratio (2)/(4)
Normal load rating	8.5' from support	E60	E60	58	60	103%	100%
Maximum load rating	8.5' from support	E60	E60	89	92	67%	65%

Table 58 - Recommended section modulus (demand over capacity less than 80%)

Location	Current S(bot)* in^3	Recommended S(bot) in^3	Recommendation
HX, Midspan, NG, T2, S3	2,350.0	3,080.0	Add 1.13 in. thickness cover plate to the bottom
HX, Midspan, SG, T2, S3	2,250.0	3,080.0	Add 1.28 in. thickness cover plate to the bottom
HX, Midspan, NG, T1, S3	2,334.0	3,080.0	Add 1.15 in. thickness cover plate to the bottom
HX, Midspan, SG, T1, S3	1,938.0	3,080.0	Add 1.75 in. thickness cover plate to the bottom
HX, Midspan, NG, T2, S2	1,977.0	3,080.0	Add 1.7 in. thickness cover plate to the bottom
HX, 8.5' from support NG, T2, S2	1,210.0	1,860.0	Add 1 in. thickness cover plate to the bottom
ML 15.14, FB20	64.2	**	Since long holes occurs at web of FB 20 and FB21 near the connection with G5, and 100% loss of end section of FB 19 at connection with cross girder E4-5 (Polytran Engineering Associates, P.C., 2007), it is recommended that FB 19, 20, and 21 be replaced.
Raritan Valley Line, Midspan, 40' girder	1,145.2	1,380.0	Add 0.33 in. thickness cover plate to the bottom
River Draw, 24.5' from support, 88' girder	3,938.0	4,600.0	Add 1 in. thickness cover plate to the bottom
River Draw, Midspan, 88' girder	4,924.0	5,450.0	Add 1 in. thickness cover plate to the bottom

*S(bot) = Section properties based on validated FE model

** No recommended S(bot) since it is recommended to replace FB20

Repair Recommendation with Speed Restriction

If speed restriction is applied, the impact-factor would be smaller (AREMA 2010, Article 7.3.3.3).[9] The RT recommends that the 10-mph speed restriction would be applied for HX Drawbridge. Therefore, the dynamic factor decreased to 1.268. The load ratings for critical members of the HX Draw Bridge, assuming a speed reduction is utilized, are summarized as shown in Table 59 through Table 63.

Table 59 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 2 in Span 3) at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	8.5' from support	E60	77	78%
	Midspan*	E57	65	88%
Maximum load rating	8.5' from support	E60	115	52%
	Midspan*	E57	97	59%

*Controls

Table 60 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 1 in Span 3) at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	Midspan	E57	64	89%
Maximum load rating	Midspan	E57	97	59%

Table 61 - Rating results for Bergen County MP 5.48 using as-inspected section properties (south girder under Track 2 in Span 3) at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	8.5' from support	E60	81	74%
	Midspan*	E57	62	91.9%
Maximum load rating	8.5' from support	E60	121	49.6%
	Midspan*	E57	93	61.3%

*Controls

Table 62 - Rating results for Bergen County MP 5.48 using as-inspected section properties (south girder under Track 1 in Span 3) at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	Midspan	E57	52	110%
Maximum load rating	Midspan	E57	77	74.0%

Table 63 - Rating results for Bergen County MP 5.48 using as-inspected section properties (north girder under Track 2 in Span 2) at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	8.5' from support	E60	66	90.9%
	Midspan*	E57	54	106%
Maximum load rating	8.5' from support	E60	98	61.2%
	Midspan*	E57	80	71.2%

*Controls

As shown in Table 59 through Table 63, after restricting the speed to 10mph, the midspan section of south girder under Track 1 in Span 3 and the midspan section of the north girder under Track 2 in Span 2 still do not satisfy the demand while other members are satisfied. Therefore, section improvements as shown in Table 64 are recommended.

Table 64 - Recommended section modulus after speed restriction (demand over capacity less than 80%)

Location	**Current S(bot)* in^3**	**Recommended S(bot) in^3**	**Recommendation**
Midspan, south girder under Track 1 in Span 3	1,938	2,570	Add 0.375 in. thickness cover plate to the bottom
Midspan, north girder under Track 2 in Span 2	1,977	2,570	Add 0.365 in. thickness cover plate to the bottom

*S(bot) = Section properties based on FE model

If speed restriction is applied to Raritan Valley Line MP31.15, the RT recommends a 10-mph speed restriction be applied. Therefore, the dynamic factor changes to 1.274. The load ratings for critical members are summarized in Table 65. The demand-over-capacity percentage of normal rating is 81.6%. Therefore, No major repair is recommended for this structure.

Table 65 - Rating results for Raritan Valley Line MP31.15 using as-inspected section properties at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	Midspan	E62	76	81.6%
Maximum load rating	Midspan	E62	114	54.4%

If speed restriction is applied for North Jersey Coast Line MP 0.39, the RT recommends that a 10-mph speed restriction be applied. Therefore, the dynamic factor changes to 1.247. The load ratings for critical members are summarized In Table 66. The highest demand-over-capacity percentage for normal rating is 87%. Therefore, the RT recommends a 0.5-in. cover plate to be added to the bottom of the girder, as shown in Table 67 to satisfy a demand-over-capacity percentage of less than 80 percent.

Table 66 - Rating results for River Draw using as-inspected section properties at 10mph

As Inspected		Equivalent Cooper E Load for 286-kips Rail Car	Cooper E Rating	Comparison
Rating Type	Location	FE Model (1)	FE Model (3)	Demand over Capacity Ratio (1)/(3)
Normal load rating	24.5' from support	E60	69	87.0%
	Midspan*	E47	58	81.0%
Maximum load rating	24.5' from support	E60	109	55.0%
	Midspan*	E47	91	51.6%

Table 67 - Recommended section modulus for River Draw after speed restriction (demand over capacity less than 80%)

Location	**Current S(bot)* in^3**	**Recommended S(bot) in^3**	**Recommendation**
River Draw, 24.5' from support, 88' approaching span	3,938.0	4,250.0	Add 0.5 in. thickness cover plate to the bottom

*S(bot) = Section properties based on validated FE model

<u>Repair Cost for Accommodating 286-kips Rail-Car Loading</u>

Based on the repair recommendations that were made based on field-testing and FE analysis, the repair cost of accommodating 286-kips rail car loading was analyzed, and the results are summarized in Table 68. No speed restriction was considered for the repair cost analysis.

Please note that the structural information, such as section properties and section losses, were extracted from the latest inspection reports provided by NJ Transit. Since additional section losses or gains might occur after the latest inspection of the bridges, the repair cost needs to be evaluated before further use.

Table 68 - Repair cost for accommodating 286-kips rail-car loading

Bridge	Alternative	Description	Repair Cost
HX	Cover Plate—100% D/C Ratio	Add steel cover plates to the bottom flanges of existing through girders	$2.92 million
HX	Cover Plate—80% D/C Ratio	Add steel cover plates to the bottom flanges of existing through girders	$5.88 million
MP 15.14	Cover Plate—100% D/C Ratio	Various structural steel repairs to floorbeams and girders	$0.98 million
Raritan Valley MP 31.15	Cover Plate—80% D/C Ratio	Add steel cover plates to the bottom flanges of existing through girders	$0.86 million
North Jersey Coast MP 0.39	Cover Plate—80% D/C Ratio	Add steel cover plates to the bottom flanges of existing through girders	$9.63 million

NOTES:
1) The cost estimates above would need to be adjusted to include incidental site preparation work.
2) All estimates will be further refined with additional engineering analysis.
3) D/C = demand over capacity.

Repair Recommendations and Cost Analysis Based On Load Rating Using AREMA Specifications

According to the load rating using AREMA Specifications presented in the previous results sections, the RT recommends the following improvement as shown in Table 69 in order to make the critical sections satisfy the same load rating criteria listed in the "repair recommendations based on load rating using FE model" section. The estimated cost to achieve this improvement is also listed in the following table. According to the load rating results, the repair recommendations based on load rating using AREMA for Bergen County Line MP 5.48 (HX Draw) is same as the repair recommendations based on load rating using FE model, therefore the repair recommendations and repair cost for this bridge is not listed in this section.

Table 69 - Repair recommendations and cost estimation based on load rating using AREMA Specifications

Bridge	Alternative	Recommendation	Repair Cost
Main Line MP 15.95	Cover Plate — 100% D/C Ratio	Add 0.15" steel cover plates to the bottom flanges of center girders	$ 0.53 million
Main Line MP 15.95	Cover Plate — 80% D/C Ratio	Add 0.5" steel cover plates to the bottom flanges of center girders	$ 0.57 million
Main Line MP 15.14	Cover Plate — 100% D/C Ratio	It is recommended that FB 19, 20, and 21 be replaced and add 0.5" steel cover plate to the bottom flange of G30 to G39	$ 1.7 million
Main Line MP 15.14	Cover Plate — 80% D/C Ratio	It is recommended that FB 19, 20, and 21 be replaced, add 0.4" steel cover plates to the bottom flange of center girder under active track and add 1.5" steel cover plate to the bottom flange of G30 to G39	$ 3.72 million
Raritan Valley MP 31.15 (Middle Brook)	Cover Plate — 100% D/C Ratio	Add 0.22" steel cover plates to the bottom flanges of center girders	$ 0.77 million
Raritan Valley MP 31.15 (Middle Brook)	Cover Plate — 80% D/C Ratio	Add 0.55" steel cover plates to the bottom flanges of center girders	$ 1.01 million
North Jersey Coast MP 0.39 (River Draw)	Cover Plate — 100% D/C Ratio	Add 0.6 in. thickness cover plate to the bottom of section B of girders (88' approach span)	$ 4.17 million
North Jersey Coast MP 0.39 (River Draw)	Cover Plate — 80% D/C Ratio	Add 1 in. thickness cover plate to the bottom of girders and 0.6 in. more thickness cover plate to the bottom of section B of girders (88' approach span)	$ 12.53 million

Summary of Repair Cost for Accommodating 286-kips Rail-Car Loading

The previous two sections discussed two kinds of repair recommendations and estimated cost to achieve concluded improvements based on load rating using FE model (FE model method) and AREMA Specifications (AREMA Specifications method), respectively. In this section, the RT summarized the repair cost of two kinds of repair recommendations. As shown in Table 70, two alternatives are provided: the demand over capacity ratio of 100% and the demand over capacity ratio of 80%. From the estimated cost, the AREMA Specifications method is higher than the FE model method. If we selected the 100% D/C Ratio standard and using FE model method, we can obtain the minimum repair cost needed which is totally $ 3.9 million. If 80% D/C Ratio standard and AREMA Specifications method applied, we can get the maximum repair cost which is totally $ 21.55 million.

Table 70 - Summary of Repair Cost

Bridge	Alternative: 100% D/C Ratio		Alternative: 80% D/C Ratio	
	Repair Cost (FE Model)	Repair Cost (AREMA)	Repair Cost (FE Model)	Repair Cost (AREMA)
Main Line MP 15.95	No upgrade	$ 0.53 million	No upgrade	$ 0.57 million
Main Line MP 15.14	$ 0.98 million	$ 1.70 million	$ 0.98 million	$ 3.72 million
Bergen County Line MP 5.48 (HX Draw)	$ 2.92 million	$ 2.92 million	$ 5.88 million	$ 5.88 million
Raritan Valley Line MP 31.15 (Middle Brook)	No upgrade	$ 0.77 million	$ 0.86 million	$ 1.01 million
North Jersey Coast Line MP 0.39 (River Draw)	No upgrade	$ 4.17 million	$ 9.63 million	$ 12.53 million
Total Cost	$ 3.90 million	$ 9.37 million	$ 17.35 million	$ 21.55 million

COST–BENEFIT ANALYSIS FOR HX DRAWBRIDGE FREIGHT TRAFFIC

Introduction

The RT conducted a benefits analysis for raising the weight restriction on the NJ Transit HX Drawbridge to 286,000 lbs, using available data. Currently, the bridge only supports weights of 263,000 lbs. per rail car, and raising this restriction is expected to result in an increase in the weights transported by freight rail cars using this line.

The HX Drawbridge lies along the Bergen Branch of NJ Transit's Main and Bergen County (M&B) Line. Incoming freight traffic to New Jersey from out-of-state arrives via Class I Norfolk-Southern (NS) Railway to a rail yard in Croxton, New Jersey. Freight cars are then transferred from the NS line to NJ Transit's M&B Line, using the HX Drawbridge, for distribution to customers and destinations throughout Northern New Jersey and New York State. Similarly, return trips are aggregated from destinations off the M&B Line, and transferred over the HX Drawbridge to Croxton rail yard. Figure 77 provides a map of the New Jersey railway network, overlaid on the highway network for context, with the locations of Croxton and the HX Drawbridge.

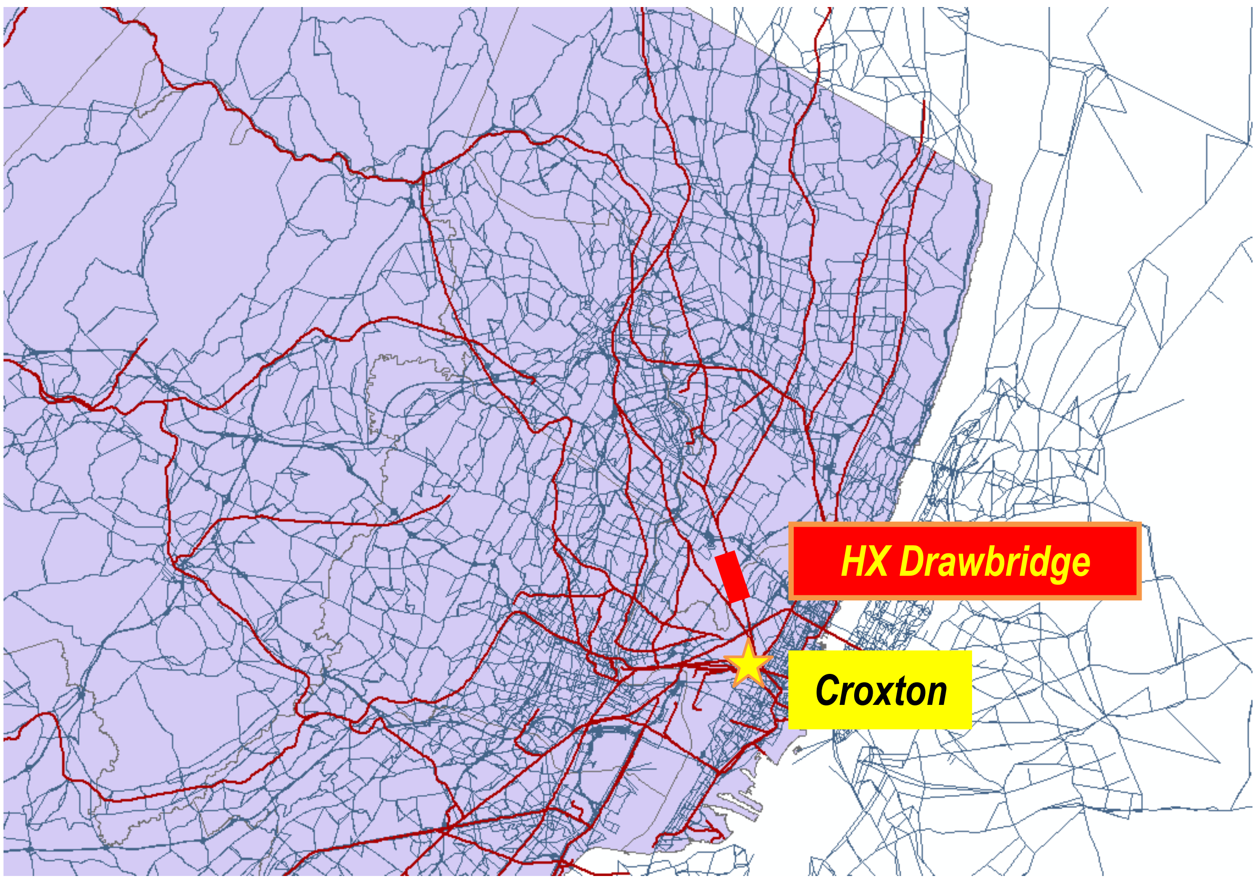


Figure 77. Location of HX Drawbridge in NJ State railway network

Based on the literature, one of the key benefits of the 286-kips rail standard is an increase in economic competitiveness caused by larger, heavier shipments, and a resulting decrease in unit cost for transport of both raw materials and processed goods. The RT confirmed this based on interviews with freight rail experts at NJDOT and customers using freight rail shipments.

Case Study: Bay State Milling

Bay State Milling, a flour mill located in Clifton, New Jersey, produces 14,000 hundredweights (cwt.) of product daily, generating $80–100 million in sales annually. The company brings in wheat from out-of-state via NS Railway and the HX Drawbridge to their facility in Clifton. Bay State Milling alone is responsible for more than 50 cars per week, or over 2,400 cars per year, based on information provided by the company. According to data provided by NJ Transit for the period of July 1 through December 31, 2010, Bay State Milling rail traffic accounted for 30 percent of all rail traffic using the HX Drawbridge.

Companies such as Bay State Milling that receive the bulk of their goods from freight rail stand to gain from an increase to 286-kips standards for rail cars. In a statement from the company, they indicate that "The Company's inability to utilize 286K cars greatly restricts its ability to compete against flour mills in Pennsylvania and New York that ship into the company's market area. With the majority of country elevators now located along modern 286K-capable lines, loading larger, newer-generation cars, Bay State is now restricted from the cash markets and must source from only those elevators still willing to light load. We pay a premium for this and face a shrinking supply source."

Due to these restrictions, planners at both NJDOT and NJ Transit have indicated that it is possible for Bay State and other like companies to move to locations where shipments are not restricted to 263,000 lbs., thus gaining an economic advantage. Based on this hypothesis, the RT studied the potential negative impacts that Bay State Milling closing its New Jersey flour mill and moving to a location out-of-state would have on New Jersey.

The economic effects of a company such as Bay State Milling moving out-of-state are tremendous. According to the company, the Clifton mill employs over 40 people at a payroll of over $2.5 million annually, and pays property taxes of $550,000 per year. A loss of an entity of this size would have a significant impact on the local and state economies. Furthermore, relocation to an out-of-state site has negative impacts on the state's highway and transportation networks. Bay State Milling distributes its product to customers located throughout New Jersey and downstate New York, as well as a small number across the Eastern Seaboard. Currently, these deliveries are made from Clifton via truck on New Jersey's roadways. If the mill were to move to an out-of-state location, it would be located further away from the majority of its customer base in New Jersey. This would result in more truck-miles on New Jersey's roads, resulting in a negative impact resulting from increased maintenance costs, congestion, and pollution. Finally, relocation would most likely result in a loss in revenue for the freight rail operators, including NS, who would carry their shipments fewer miles.

The RT thus investigates the potential negative impact of Bay State's flour mill moving out-of-state by measuring the increase in truck travel distances. The team has also attempted to quantify the resulting negative impacts of the increase in truck-miles, presenting a total cost of the loss of Bay State Milling. This provides insight into the potential benefit of raising the weight standard on the HX Drawbridge to 286,000 lbs. Finally, a discussion is presented on extending this analysis to all shipments using the HX Drawbridge, and throughout the State of New Jersey.

Methodology

There is a potential that Bay State Milling will move out of state, with the most likely destinations being in northeastern Pennsylvania or the lower reaches of Upstate New York, based on those location's proximity to Northern New Jersey. Bay State Milling delivers their finished product (flour) and byproducts to customers mostly located in New Jersey and downstate New York, as well as others throughout the Eastern Seaboard. For the purposes of this analysis, deliveries to locations not within New Jersey or downstate New York (Westchester County, New York City, and Long Island) are excluded.

The benefit of Bay State remaining in Clifton, New Jersey, is calculated as the difference between the truck trips currently originating in Clifton to their destinations, and the difference between truck trips potentially originating out-of-state, to their destinations. Delivery data was provided by Bay State Milling for the period of July 1 through December 31, 2010, corresponding to the freight rail data obtained from NJ Transit. This data set identified the number, frequency, and weights of Bay State's truck deliveries, and the destinations of the deliveries (approximate location of customers). With the origin-destination information at hand, the RT then identified the shortest path between the origin and destination using the k-shortest path algorithm developed by Ozbay et al. applied to the North Jersey Regional Transportation Model–Enhanced (NJRTM-E).[32] NJRTM-E (shown in Figure 78) is a regional travel demand model developed by the North Jersey Transportation Planning Authority (NJTPA) representing all highway infrastructures in Northern New Jersey. The model predicts travel in the region and assigns vehicular flows to the network to estimate volumes, travel times, speeds, and the like, for all links in the network.

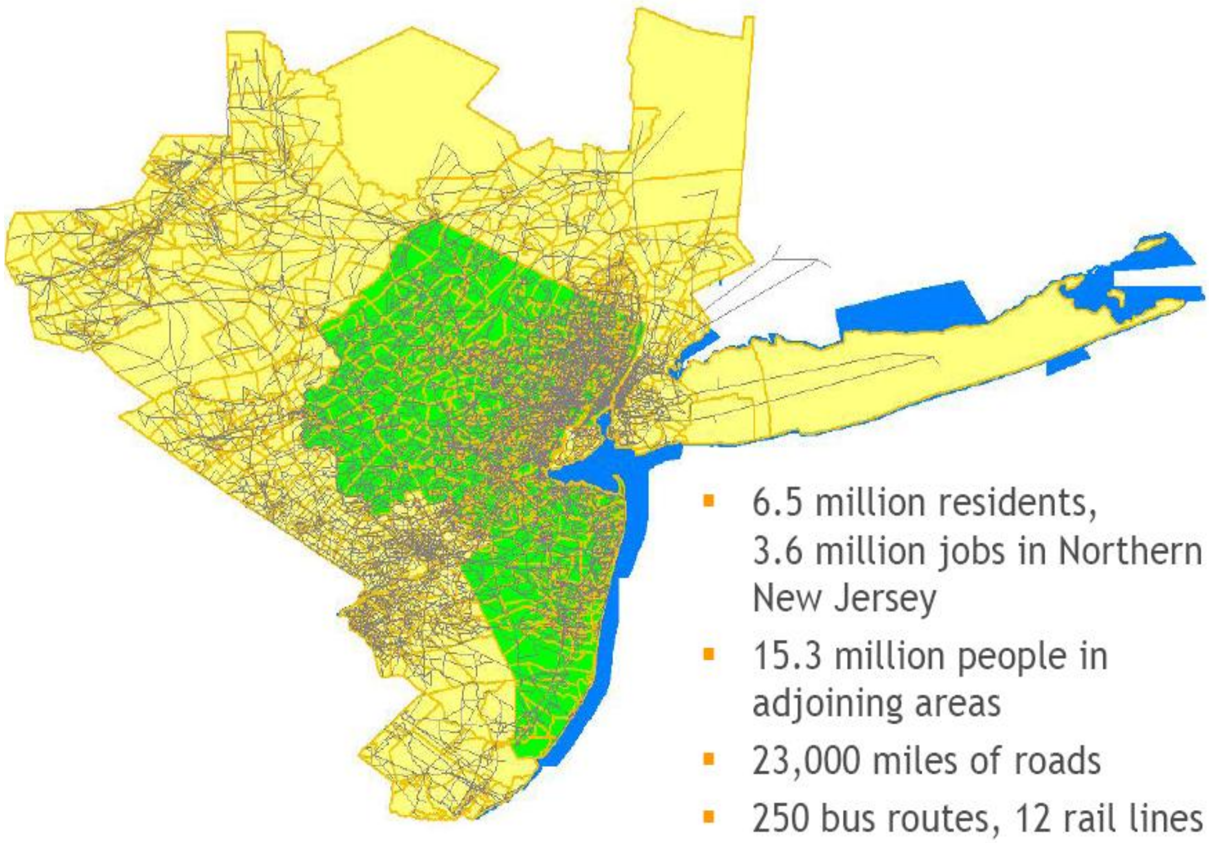


Figure 78. North Jersey Regional Transportation Model–Enhanced network(33)

This model is used to estimate current and potential future truck trips along New Jersey's highway network by estimating the probable shortest path (in this context, "shortest path" refers to the route with the lowest total travel time) between an origins–destination (O-D) pair. An example of the k-shortest path algorithm used to identify shortest routes for truckers between two points is shown in Figure 79. In this example, Croxton, New Jersey, is selected as an origin, with Clifton, New Jersey, as the destination, and the route highlighted in the map representing the shortest path for a truck to take between the two destinations.

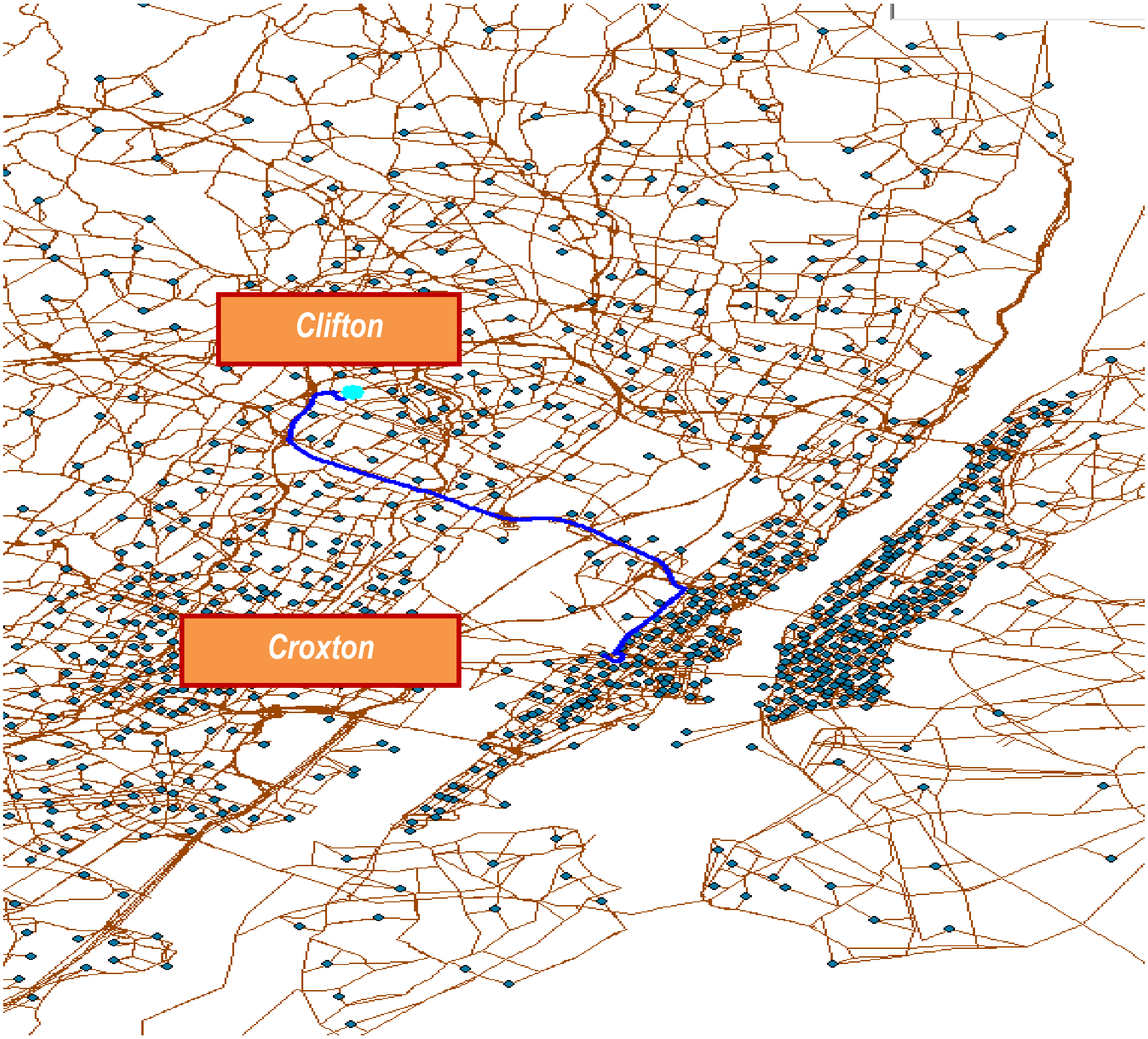


Figure 79. Example of shortest path identification shown on the highway network

For the analysis, routes between O-D pairs (origin in Clifton, destination at customer location) are traced and scaled by the frequency and size of deliveries to determine total truck impact currently made by Bay State Milling's deliveries. For a potential future case, the same methodology is repeated, with uncertainty in the origin of the trips, however. Rather than Clifton, if the mill moves out-of-state the goods enter New Jersey from an unknown location. Based on the most likely locations of northeastern Pennsylvania and lower New York State, the three-most-probable entry points to New Jersey are selected: the New Jersey–Pennsylvania border via I-78, the New Jersey–Pennsylvania border via I-80, and the New Jersey–New York border via I-287/NJ Route 17. These entry points are identified in Figure 80. O-D trips are then traced from each of these three potential entry points to determine the routes taken within New Jersey. The dataset includes goods that were picked up from the Clifton mill combined with local deliveries made in Clifton. So, in the analysis, the destination for all of these goods was assumed to be Clifton, New Jersey. Finally, the difference between the current trips and potential future trips from out-of-state is subtracted to measure total impact, or benefits.

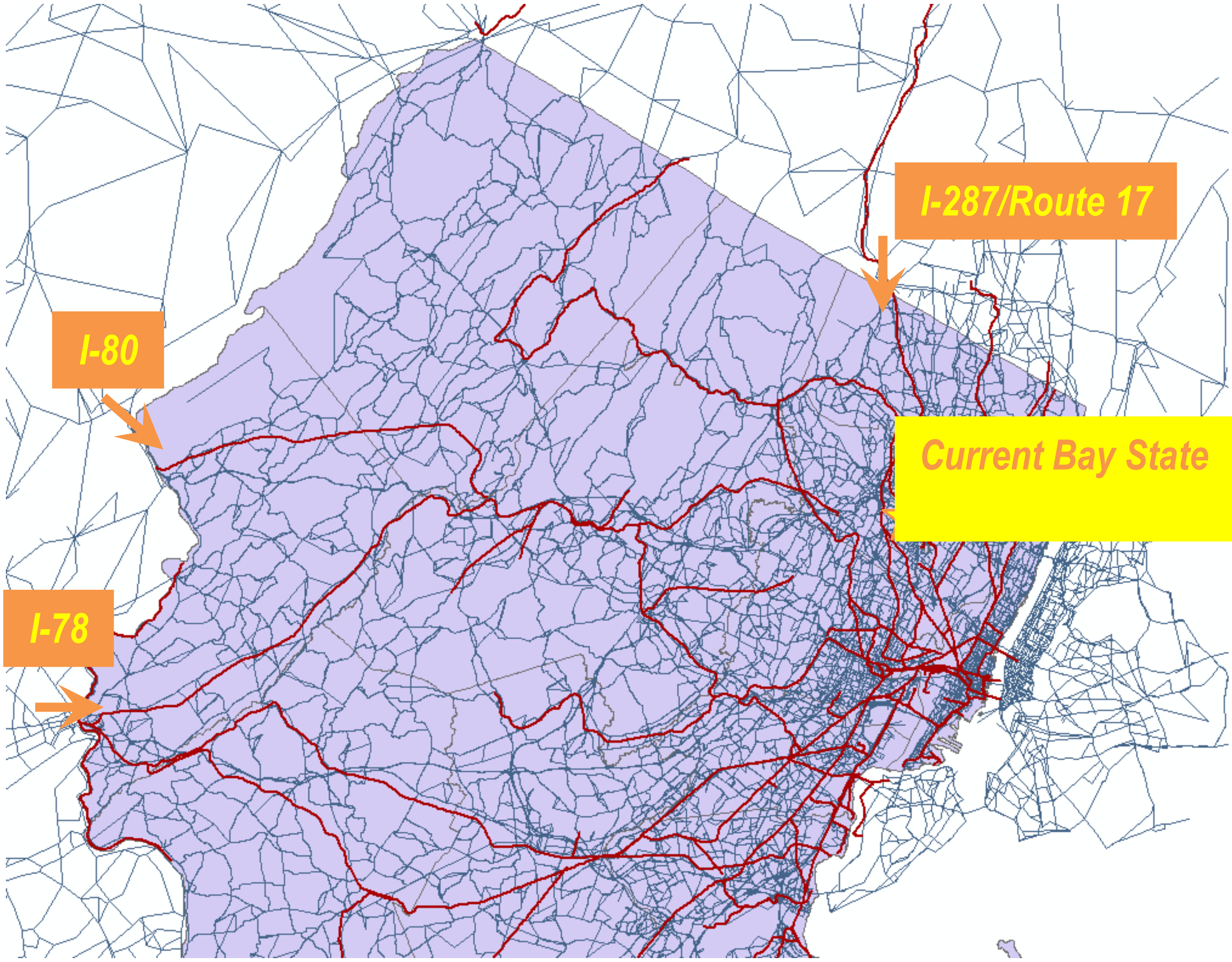


Figure 80. Assumed New Jersey entry points of out-of-state deliveries

Quantification of Trucking and Freight Rail Costs

Literature on the estimation of freight costs on rail or truck is limited. Forkenbrock conducted two studies on estimation of unit costs of trucking and freight rail per ton-mile.[34,35] Table 71 presents the unit costs calculated by Forkenbrock per ton-mile. The research identified different cost categories, including the private cost of trucking, as well as the external and social costs imposed by trucks on roadways and roadway users. This includes accidents and pollution.

Table 71 - Costs of trucking in 1994 cents per ton-mile from Forkenbrock[34]

Type of Cost	**Amount**
Private cost	8.42
External cost	
Accidents	0.59
Air pollution	0.08
Greenhouse gases	0.15
Noise	0.04
User charge underpayment	0.25
Total	1.11
External cost as a percent of private cost	13.20

Ozbay et al. conducted a study focused just on travel for New Jersey.[32] They were able to develop cost functions for categories of travel and externalities for all vehicles, based on New Jersey–specific data where available. These functions are shown in Table 72 for the different cost categories shown. The RT is in an advantageous position to use this methodology, as the functions have been integrated with the NJRTM-E model to estimate functions for all links in the network, and along the O-D pair's shortest paths identified.

Table 72 - Cost functions for highway travel developed by Ozbay et al.[32]

Cost	**Total Cost Function**	**Variable Definition**	**Data Sources**
Vehicle operating	$C_{opr} = 7{,}208.73 + 0.12(m/a) + 2783.3a + 0.143m$	a =Vehicle age (years) m = Vehicle miles traveled	AAA,[36] USDOT,[37] KBB.[38]

Congestion	$C_{cong} = \begin{cases} Q.\frac{d}{V_o}.\left(1+0.15\left(\frac{Q}{C}\right)^4\right).VOT & \text{if } Q \le C \\ Q.\frac{d}{V_o}.\left(1+0.15\left(\frac{Q}{C}\right)^4\right).VOT \ + Q.\left(\frac{Q}{C}-1\right).\frac{VOT}{2} & \text{if } Q > C \end{cases}$		*Q* = Volume (veh/hr) *d* = Distance (mile) *C* = Capacity (veh/hr) *VOT* = Value of time ($/hr) V_o = Free flow speed (mph)	Mun[39] Small and Chu[40]
Accident	*Category 1*: interstate freeway	$C_{acc} = 127.5Q^{0.77}.M^{0.76}.L^{0.53} + 114.75Q^{0.85}.M^{0.75}.L^{0.49} + 198{,}900Q^{0.17}.M^{0.42}.L^{0.45}$	*Q* = Volume (veh/day) *M* = Path length (miles) *L* = no. of lanes	FHWA[41] USDOT[42]
	Category 2: principal arterial	$C_{acc} = 178.5Q^{0.58}.M^{0.69}.L^{0.43} + 18{,}359Q^{0.45}.M^{0.63}.L^{0.47}$		
	Category 3: arterial-collector-local road	$C_{acc} = 229.5Q^{0.58}.M^{0.77}.L^{0.77} + 9{,}179.96Q^{0.74}.M^{0.81}.L^{0.75}$		
Air pollution	$C_{air} = Q(0.01094 + 0.2155F)$ where $F = 0.0723 - 0.00312V + 5.403x10^{-5}V^2$		*F* = Fuel consumption at cruising speed (gal/mile) *V* = Average speed (mph) *Q* = Volume (veh/hr)	EPA[43]
Noise	$C_{noise} = 2\int_{r_1=50}^{r_2=r_{max}} (L_{eq} - 50)DW_{avg}\frac{RD}{5280}dr$ where $K = K_{car} + K_{truck}$ $K = \frac{F_c}{V_c}\left(V_c^{4.174}.10^{0.115} + 10^{5.03F_{ac} + (1-F_{ac})6.7}\right) + \frac{F_{tr}}{V_{tr}}\left(V_{tr}^{3.588}.10^{2.102} + 10^{7.43F_{atr} + (1-F_{atr})7.4}\right)$ $L_{eq} = 10log(Q) + 10log(K) - 10log(r) + 1.14$		*Q* = Volume (veh/day) *r* = distance to highway *K* = Noise-energy emis. K_{car} = Auto emission K_{truck} = Truck emission F_c = % of autos, F_{tr} = % of trucks F_{ac} =% const. speed autos F_{atr}=% of const. speed tr. V_c = Auto speed (mph) V_{tr} = Truck speed (mph) *D* = Percent discount in value per increase in the ambient noise level W_{avg} = Average home value *RD* = Residential density	Delucchi and Hsu[44]

Maint-enance	$C_M = \frac{796.32 M^{0.40} L^{0.39}}{P}$ where $P = \frac{N}{ESAL}$ $ESAL = Q \times 365 \times P_t \times T_f$	*M* =roadway length (miles) *L* = number of lanes *P* = design cycle period *ESAL* = Equivalent single axle load *N* = number of allowable repetitions (1,500,000) *Q* = Traffic volume (veh/day) P_t = Percentage of trucks in traffic T_f = Truck Factor	Ozbay et al.[(45)]

Based on these functions, costs of the O-D trips of truck deliveries are quantified, and the total benefit of Bay State Milling remaining in New Jersey is calculated.

Sensitivity Analysis

The quantification of trip costs is subject to many input assumptions that have an effect on the results. As described, the assumed entry point to New Jersey can vary between any of the three selected origins, changing the trip length and total impact. Therefore, minimum and maximum impacts are identified to present a range of values for the analysis. Additionally, the time of day of these trips is unknown, which affects travel time and quantification of external costs. For example, trips made during rush hour take longer and face more traffic than trips taken overnight. Thus, minimum and maximum values also take into account time-of-day possibilities, which are unknown.

Finally, monetization of travel time and congestion requires an assumed value of time (VOT). Based on the literature, VOT in the New York metropolitan region was assumed to be $23 per hr.[(46)] Corresponding to the average wage is a common methodology employed. The U.S. Bureau of Labor Statistics 2009 data for wages in the New York-Newark-Bridgeport area was between $26.56 per hour and $33.66 per hour. The only estimate available for commercial vehicles in the region suggests $34 per hour for light trucks to $55 per hour for semitrailers.[(47)] However, because of the uncertainty associated with the exact composition of the traffic in the entire network, the results are presented for a range of values of the composite VOT. Different assumptions of traffic composition and VOT lead to composite VOT as low as $25 per vehicle-hour and as high as $35 per vehicle-hour used in the analysis.

Table 73 presents the total difference between current trucking trip costs and potential future trucking trip costs for this case study. Data for one-half years is annualized and projected for 25 years, linearly discounted to 0 by Year 25. A 2.8 percent interest rate is also used for present value.

Table 73 - Difference in total average cost of truck trips for Bay State Milling deliveries (present versus future out-of-state origin)

Total Average Cost	Value of Time							
	$25/hr				$35/hr			
	min		*max*		*min*		*max*	
Annual benefit	$0.13	million	$0.55	million	$0.16	million	$0.71	million
Total benefit (25 years)	$1.37	million	$5.81	million	$1.74	million	$7.49	million
Per truck trip	*$ 11.56*		*$ 49.16*		*$ 14.74*		*$ 63.43*	

Table 73 presents a range of results on the total average cost of truck trips between current Bay State deliveries and future Bay State deliveries originating out-of-state. These estimates represent the total cost of trucking, and the impact of trucks on New Jersey highways. It is important to note that these numbers do not include the wage of truck drivers; or economic employment-, tax-, and toll-related measures. These calculations are strictly for routes measured and quantified according to the categories in Table 72.

Discussion

The results presented show a potential benefit of up to $7.49 million over 25 years. It is important to note that this is only for Bay State Milling's potential moving-out-of-state scenario, and measures only transportation impacts and no other significant economic measures. Also, it was noted that Bay State Milling represents 30 percent of HX Drawbridge traffic. If the same assumptions were applied to all HX traffic, the transportation-related benefits of 286-kips rail cars could be as high as $25 million over 25 years. It is also important to note that this is only one line carrying freight traffic in New Jersey.

CONCLUSIONS

In this report, a comprehensive study was performed for five typical bridges owned by NJ Transit:

(1) Main Line MP 15.95
(2) Main Line MP 15.14
(3) Bergen County Line MP 5.48 (HX Draw)
(4) Raritan Valley Line MP 31.15 (Middle Brook)
(5) North Jersey Coast Line MP 0.39 (River Draw)

For each bridge, a 3-D FE model was developed using both as-built and as-inspected section properties. Field experimental study was performed to collect the structural responses, such as strain, deflection, and velocity. The experimental data was used to validate the developed FE model and evaluate the performance of the bridge. Except for regular passenger rail cars, a typical 286-kips rail car was used to perform the tests on the HX Drawbridge at various speeds. The load rating was performed using both AREMA Specifications and FE analysis.

Based on the analysis results of this study, the following conclusions can be drawn from the results:

1) Overall, the Main Line MP 15.95 Bridge is in good condition. The load rating based on FE modeling indicates that the bridge is capable of carrying 286-kips railcar. However, based on the load rating results using AREMA's simple beam analysis, there is a need to upgrade the through girders in span 2 in order to satisfy a level of demand over capacity (D/C) ratio of 80%. Lower rating results than those obtained by the FE analysis were observed when using AREMA's Specifications because the assumed boundary conditions are pinned supports.

2) For the Main Line MP 15.14, although the rating of the critical member is low, this critical member (FB20) is under the abandoned track and will not affect the performance of the bridge directly. Inspection Report Cycle 4 implied that this bridge does not suffer much section losses and the as-inspected section moduli of the main girders under the active track is the same as the as-built properties.[20] The load rating based on the FE analysis indicates that the bridge is capable of carrying 286-kips railcar. However, the load rating results using simple beam analysis indicates the load rating of Girder 37 is larger than the D/C ratio of 100% (*e.g.* D/C ratio of normal rating for section 14.5' from support of Girder 37 is 113%). Thus, in order to satisfy the limit of D/C ratios of 100% and/or 80%, certain repairs may be needed for Girder 37. Higher load carrying capacity was observed using the FE model.

3) For the Bergen County Line MP 5.48 (HX Draw), the rating of the bridge was improved and demonstrates higher rating results than the latest inspection report after the recent repair (conducted after 2007) by NJ Transit.[21] However, in order

to safely carry 286-kips rail cars, repairs have to be made to various structural elements. Various repair alternatives were proposed, including adding cover plates to the bottom flange and limiting the maximum speed. The feasibility of repair alternatives presented in this report needs to be evaluated and reviewed by NJ Transit.

4) For the Raritan Valley Line MP 31.15, the bridge is in overall fair condition and the rating results show that the bridge is capable of carrying 286-kips rail cars. However, the ratings of some of the sections are fairly close to the limit (*e.g.* the D/C percentage is 98% for the midspan section). Therefore, repairs are needed to improve the performance of the bridge and maintain an adequate safety margin of more than 20% (i.e., D/C <80%).

5) For the North Jersey Coast Line MP 0.39 (River Draw) Bridge, similar to the Raritan Valley Line MP 31.15 Bridge, it is in an overall fair condition and the rating results show that the bridge is capable of carrying 286-kips rail cars. However, the ratings of some of the sections are fairly close to the limit (*e.g.* the D/C percentage is 97% for the section that is 24.5 ft away from the support). Therefore, repairs are needed to improve the performance of the bridge and maintain an adequate safety margin.

6) The study performed in this report was based on the latest inspection reports of each bridge and information provided by NJ Transit. However, the last inspection of these bridges was done between 2006 and 2007, which might not reflect the current and up-to-date conditions of the bridges. Therefore, the information used in this report needs to be re-evaluated and validated.

7) The fatigue analysis performed in this study indicated that the fatigue remaining life of the bridges would be reduced by a percentage of 35-50% minimum, if the 286-kips freight railcar were utilized. Thus, In order to evaluate the long term performance of the bridge and take advantage of the in-place (instrumented) sensors, further data collection and long term structural health monitoring (SHM) and evaluation of fatigue life before and during operation of the 286 railcars is recommended. This will provide important and needed information on the future use of these bridges under various types of loading.

8) The study performed in this project is focused on five typical NJ Transit bridges only. These bridges may not be representative of the remaining bridges on Amtrak, NJ Transit, and Conrail Lines. Therefore, further review and evaluation of other types of bridges is needed before extending the conclusions in this report to other bridges or other rail lines.

9) Currently, NJDOT is also considering operation of 286-kips freight cars on other lines in the state. These structures should be inspected, modeled, and load-rated to allow for 286-kips freight cars and improve the freight-rail network in New

Jersey. Maintenance, repair, and retrofit recommendations are needed to facilitate the heavier rail cars.

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APPENDIX A: EXPERIMENTAL DATA COLLECTED FROM MAIN LINE MP 15.95 BRIDGE

The experimental data of Main Line MP 15.95 is summarized in this appendix. Table 74 presents the information gathered from each experimental case, and Figure 81 through Figure 91 show the experimental data collected from the field.

Table 74 - Information about the test trains for Main Line MP 15.95

Run Number	Type	Locomotive Weight (lbs)	Distance Between Truck Centers	Axle Weight (lbs)	Speed (mph)
1	GP40-PH-2B	284,200	37’-3”	71,050	31.4
2	PL-42AC	288,000	42’-4”	72,000	30.9
3	GP40PH-20	293,650	37’-3”	73,413	37.4
4	GP40FH-2M	282,500	37’-3”	70,625	31.4

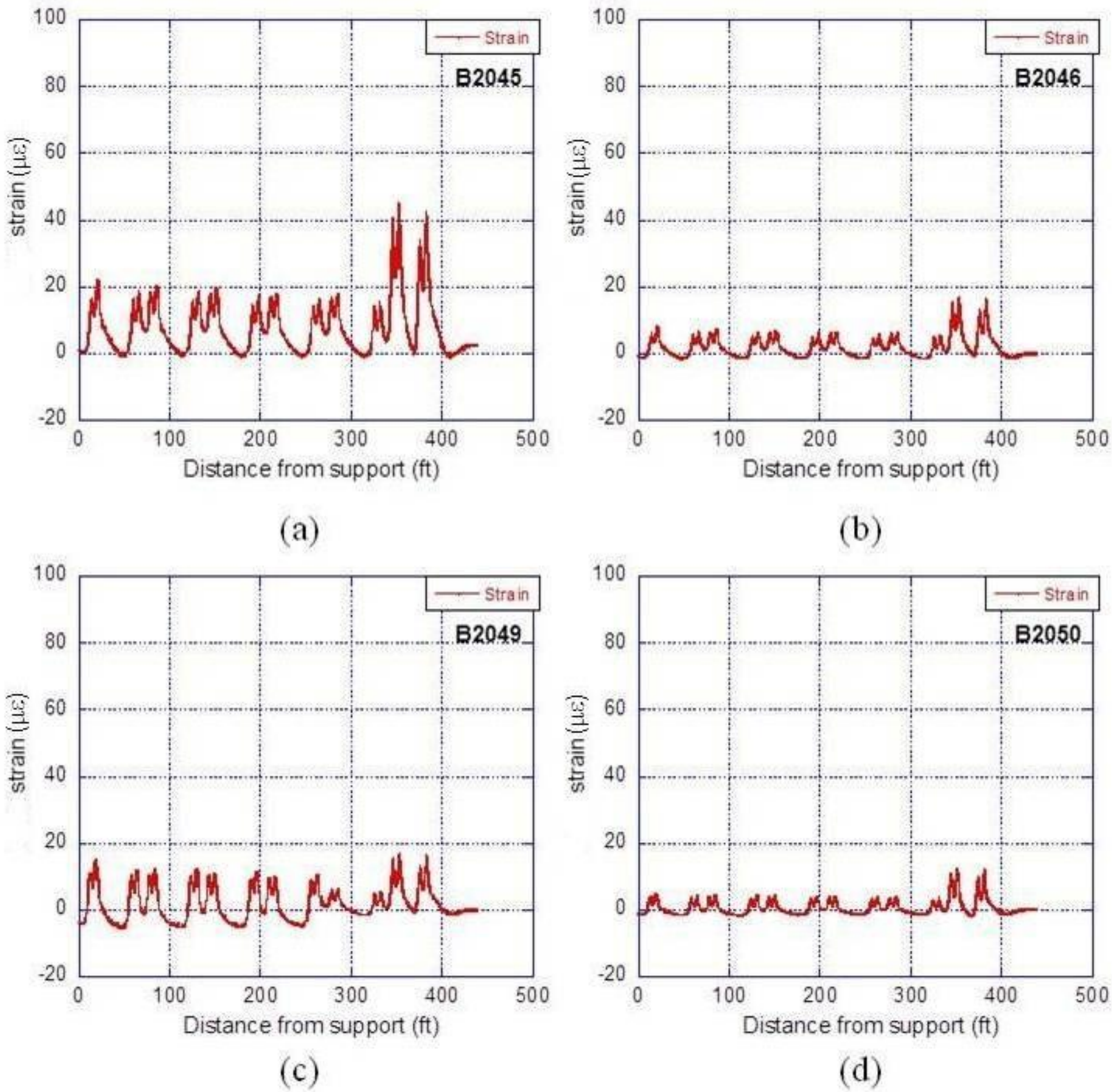


Figure 81.Strain data from run 1: *(a)* B2045,*(b)* B2046,*(c)* B2049, and*(d)* B2050

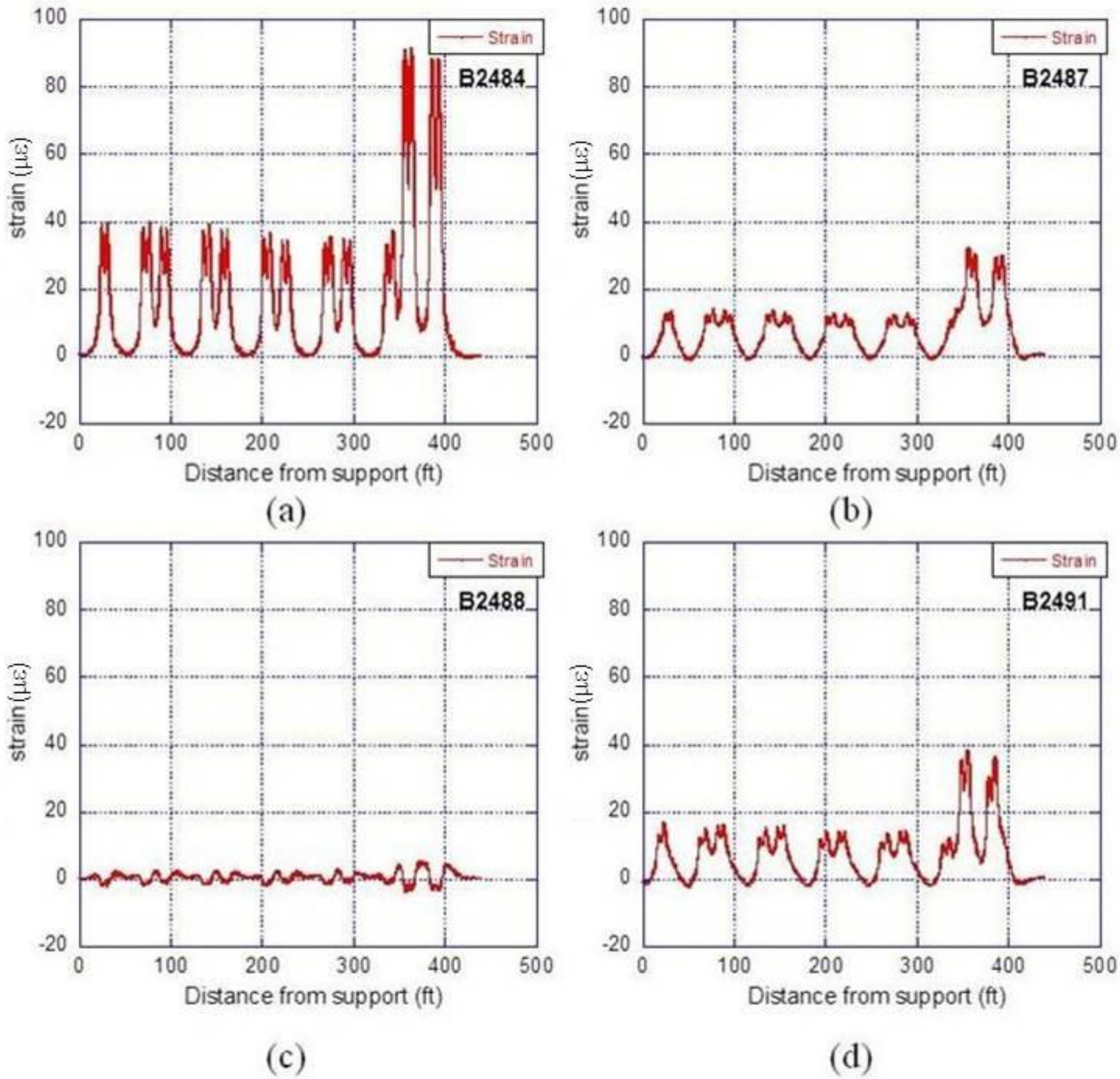


Figure 82.Strain data from run 1: *(a)* B2484,*(b)* B2487,*(c)*B2488, and*(d)*B2491

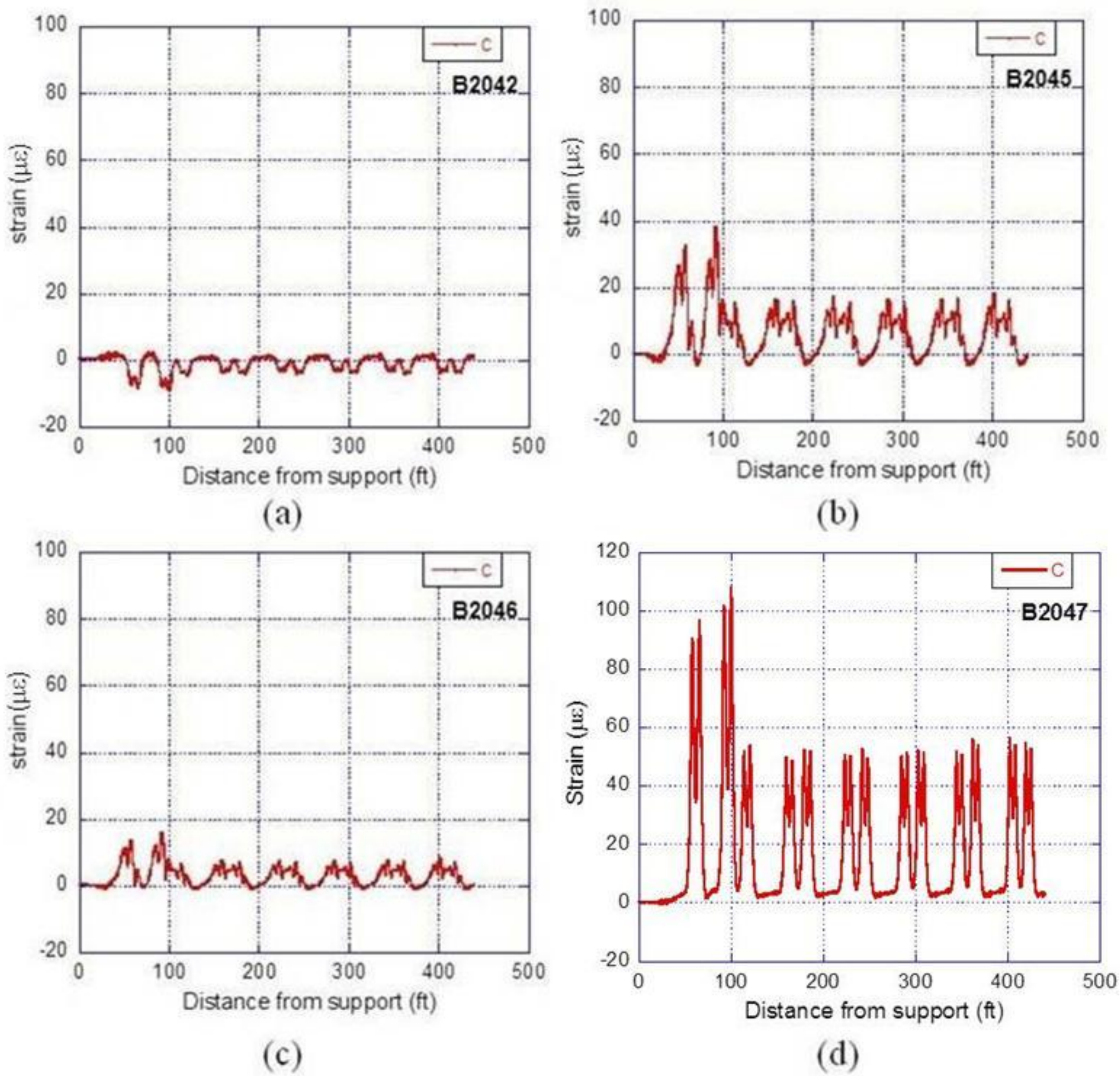


Figure 83.Strain data from run 2: *(a)* B2042,*(b)* B2045,*(c)* B2046, and*(d)* B2047

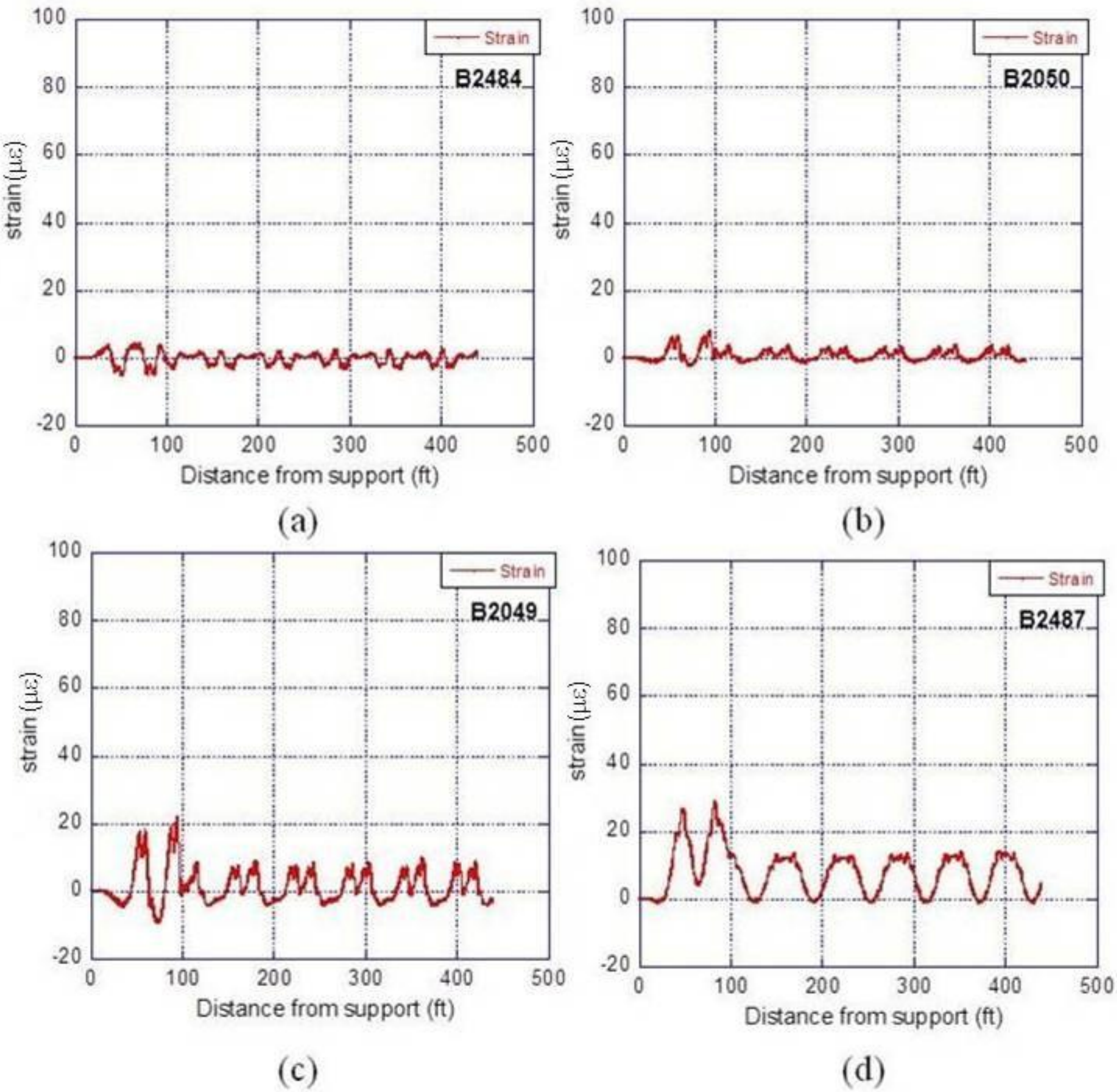


Figure 84.Strain data from run 2: *(a)* B2484,*(b)* B2050,*(c)*B2049, and*(d)*B2487

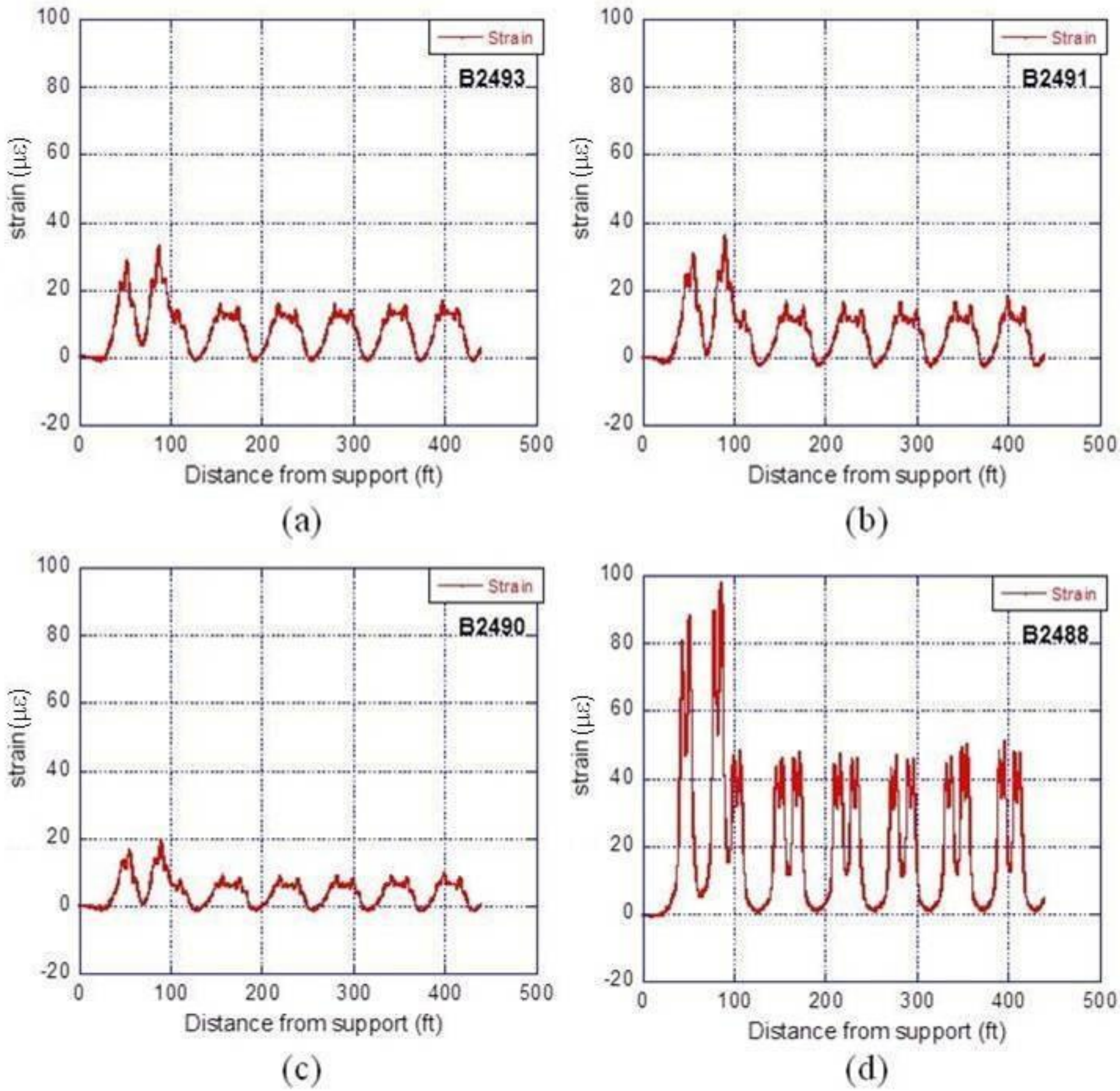


Figure 85.Strain data from run 2: *(a)* B2493,*(b)* B2491,*(c)*B2490, and*(d)*B2488

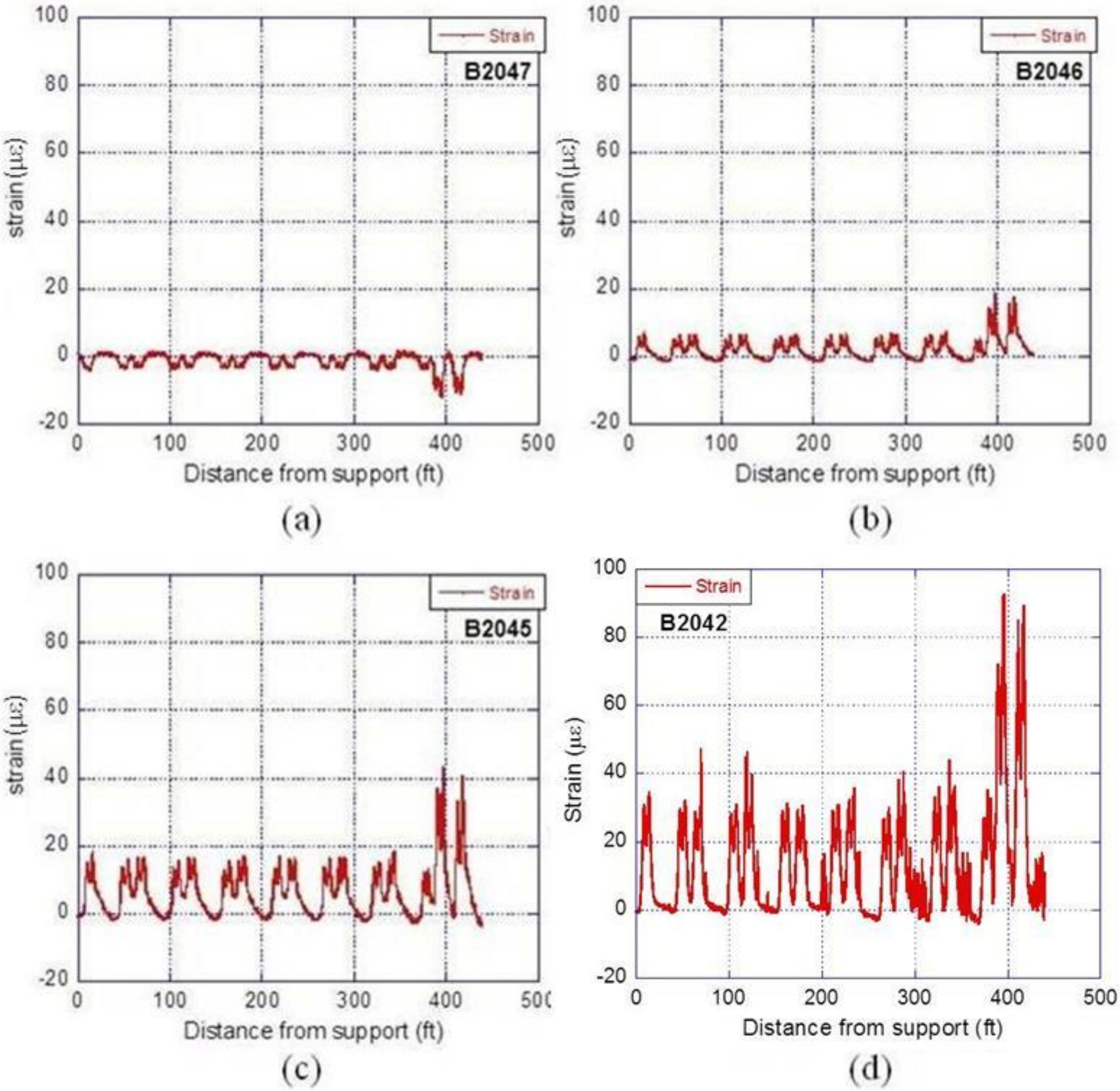


Figure 86.Strain data from run 3: *(a)* B2047,*(b)* B2046,*(c)*B2045, and*(d)*B2042

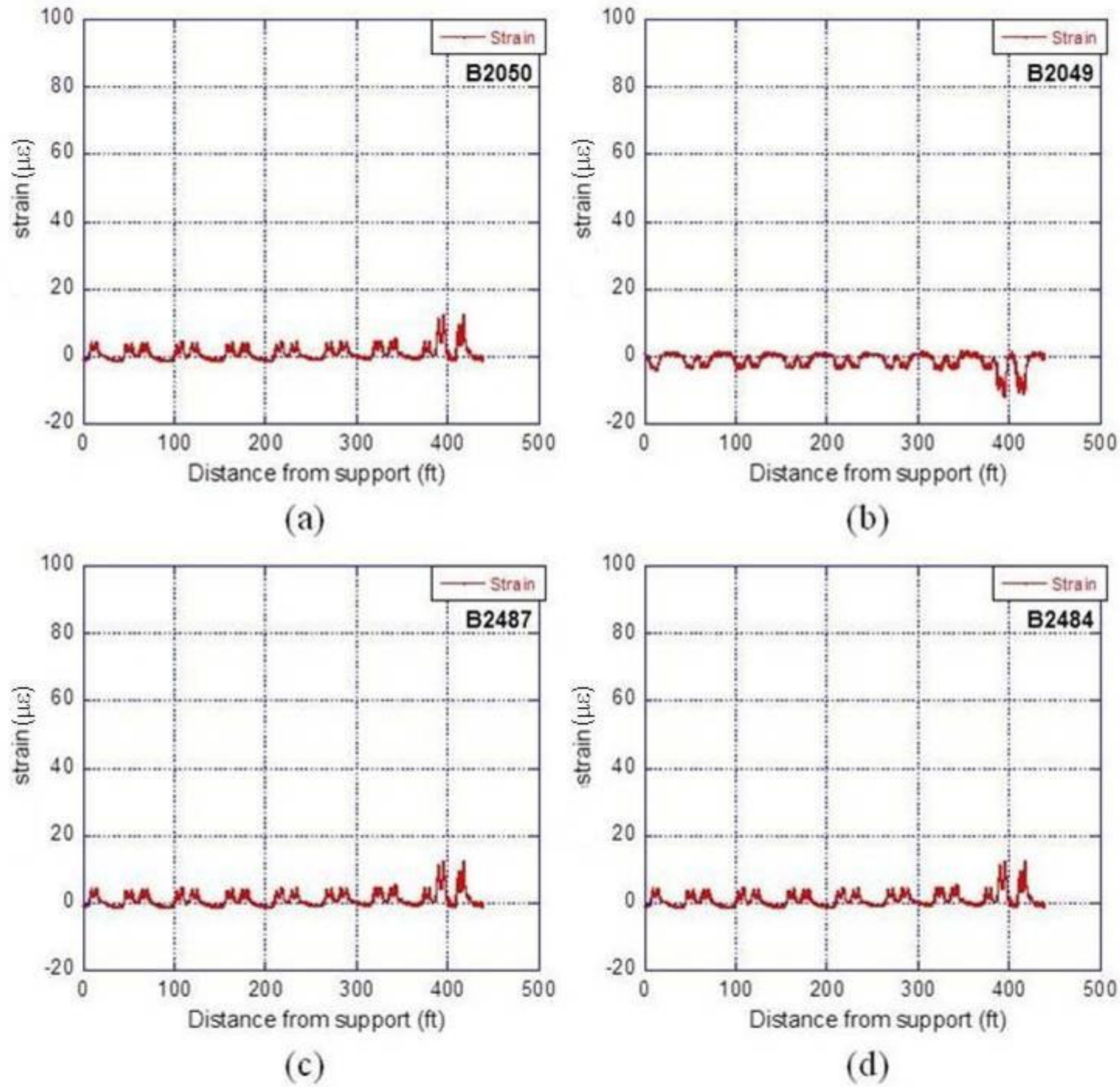


Figure 87.Strain data from run 3: *(a)* B2050,*(b)* B2049,*(c)* B2487, and *(d)* B2484

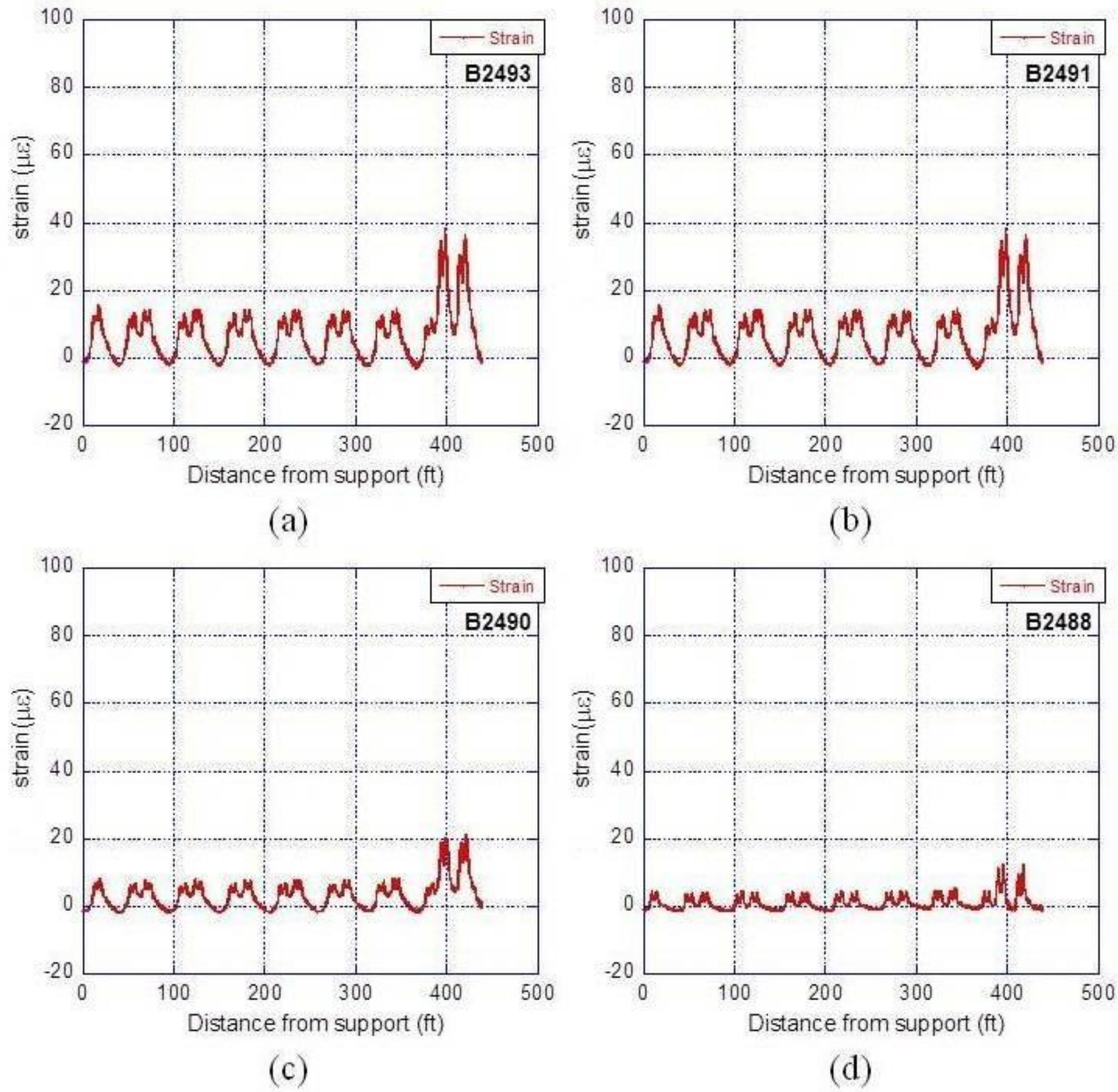


Figure 88.Strain data from run 3: *(a)* B2493,*(b)* B2491,*(c)* B2490, and*(d)* B2488

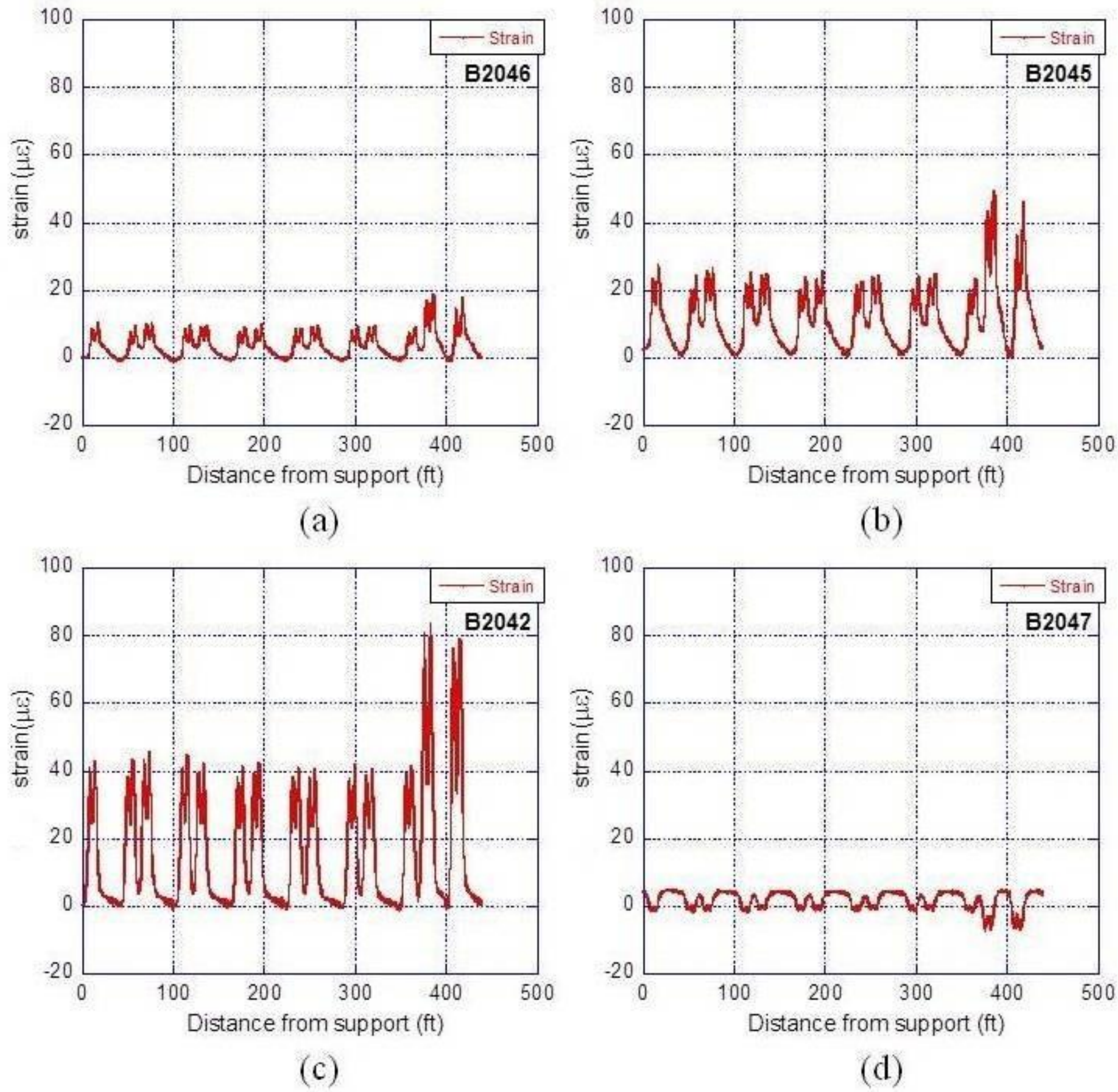


Figure 89.Strain data from run 4: *(a)* B2046,*(b)* B2045,*(c)* B2042, and*(d)* B2047

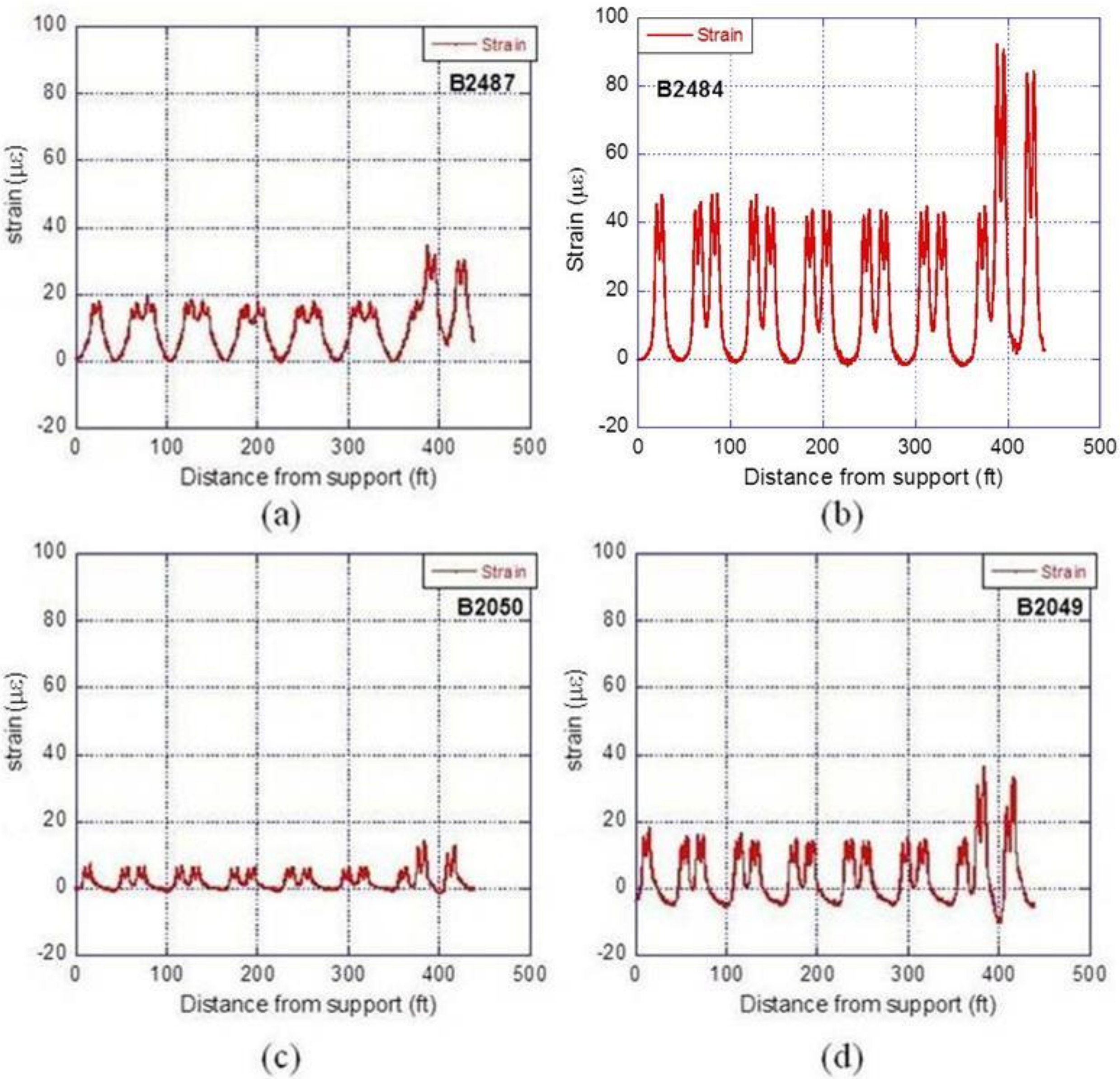


Figure 90.Strain data from run 4: *(a)* B2487,*(b)* B2484,*(c)* B2050, and*(d)* B2049

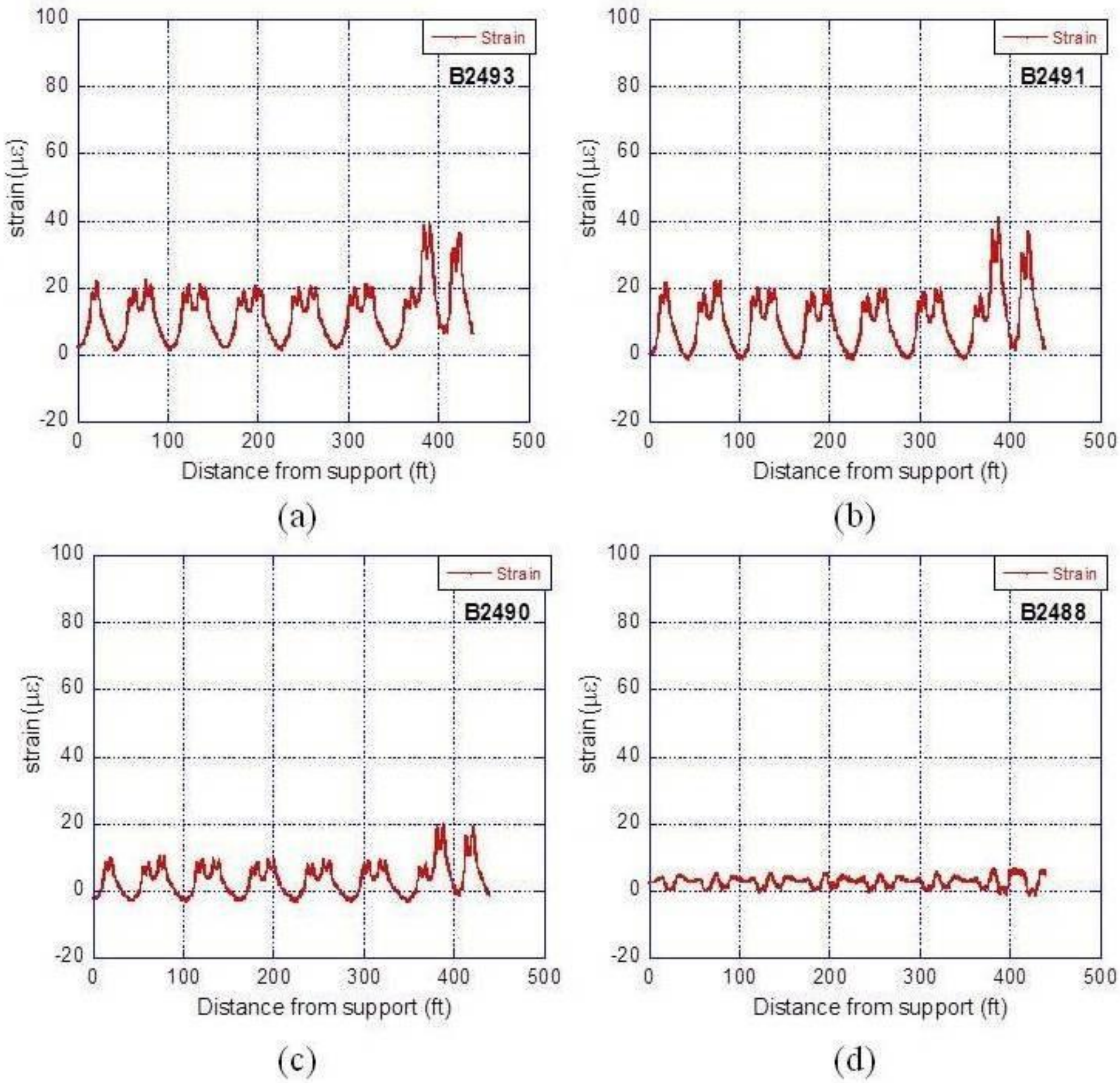


Figure 91.Strain data from run 4: *(a)* B2493,*(b)* B2491,*(c)* B2490, and*(d)* B2488

APPENDIX B: EXPERIMENTAL DATA COLLECTED FROM MAIN LINE MP 15.14 BRIDGE

The experimental data of ML MP 15.14 is summarized in this appendix. Table 75 presents the information gathered from each experimental case, and Figure 92 through Figure 101 shows the experimental data collected from field.

Table 75 - Information about the test trains for Main Line MP 15.14

Run Number	Information About the Tested Train				
	Direction	**Train Type**	**Truck Wheel Base**	**Axle Weight (lb)**	**Speed (mph)**
1	Clifton–Paterson	PL42-AC	9'-5"	72,000	31.1
2	Paterson–Clifton	F40PH-2CAT	9'	65,375	32.3
3	Clifton–Paterson	GP40FH-2M	9'	70,625	35.9

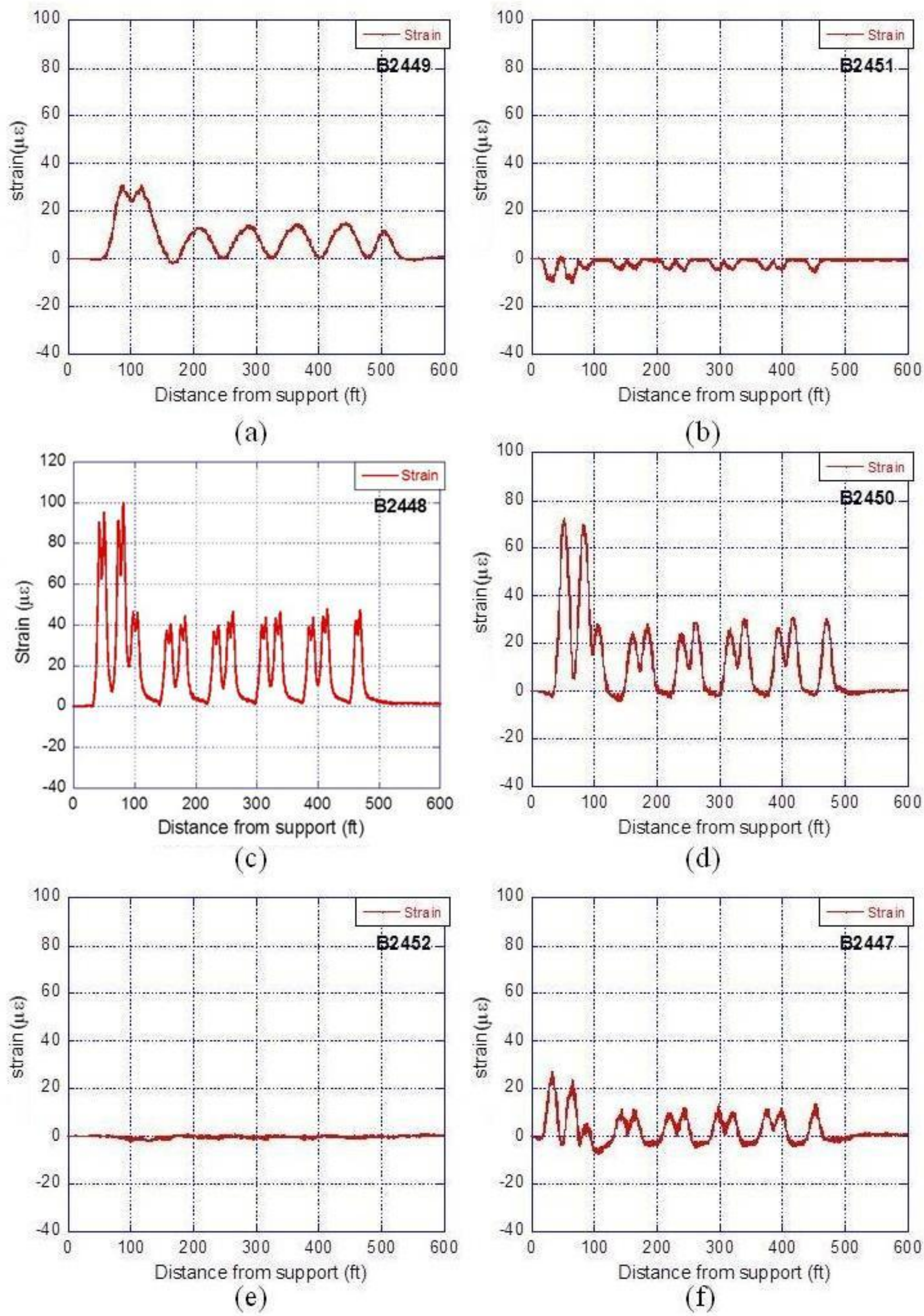


Figure 92.Strain data from run 1: *(a)* B2449,*(b)* B2451,*(c)* B2448,*(d)* B2450,(e) B2452, and*(f)* B2447

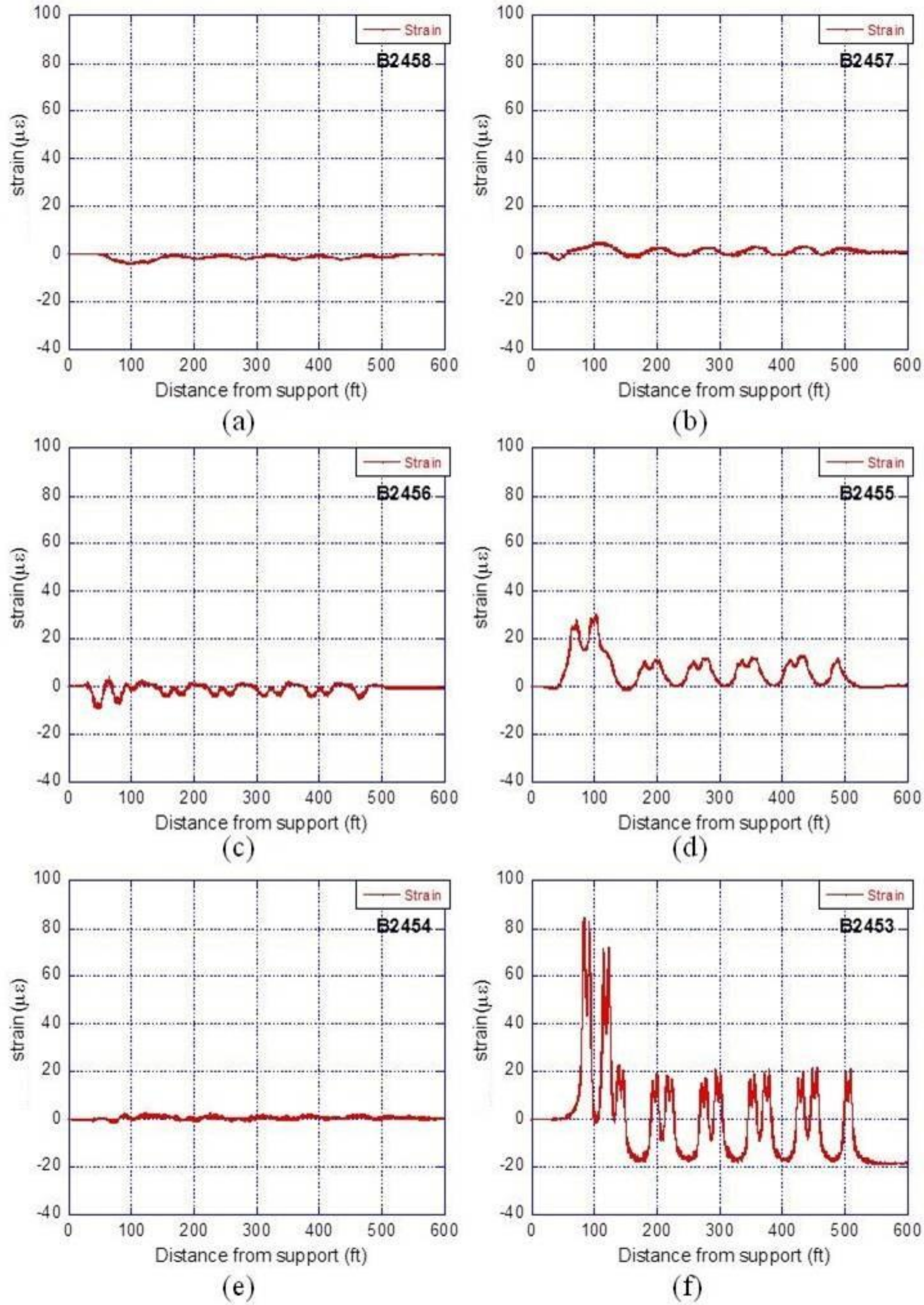


Figure 93.Strain data from run 1: *(a)* B2458,*(b)* B2457,*(c)* B2456,*(d)* B2455, *(e)* B2454, and *(f)* B2453

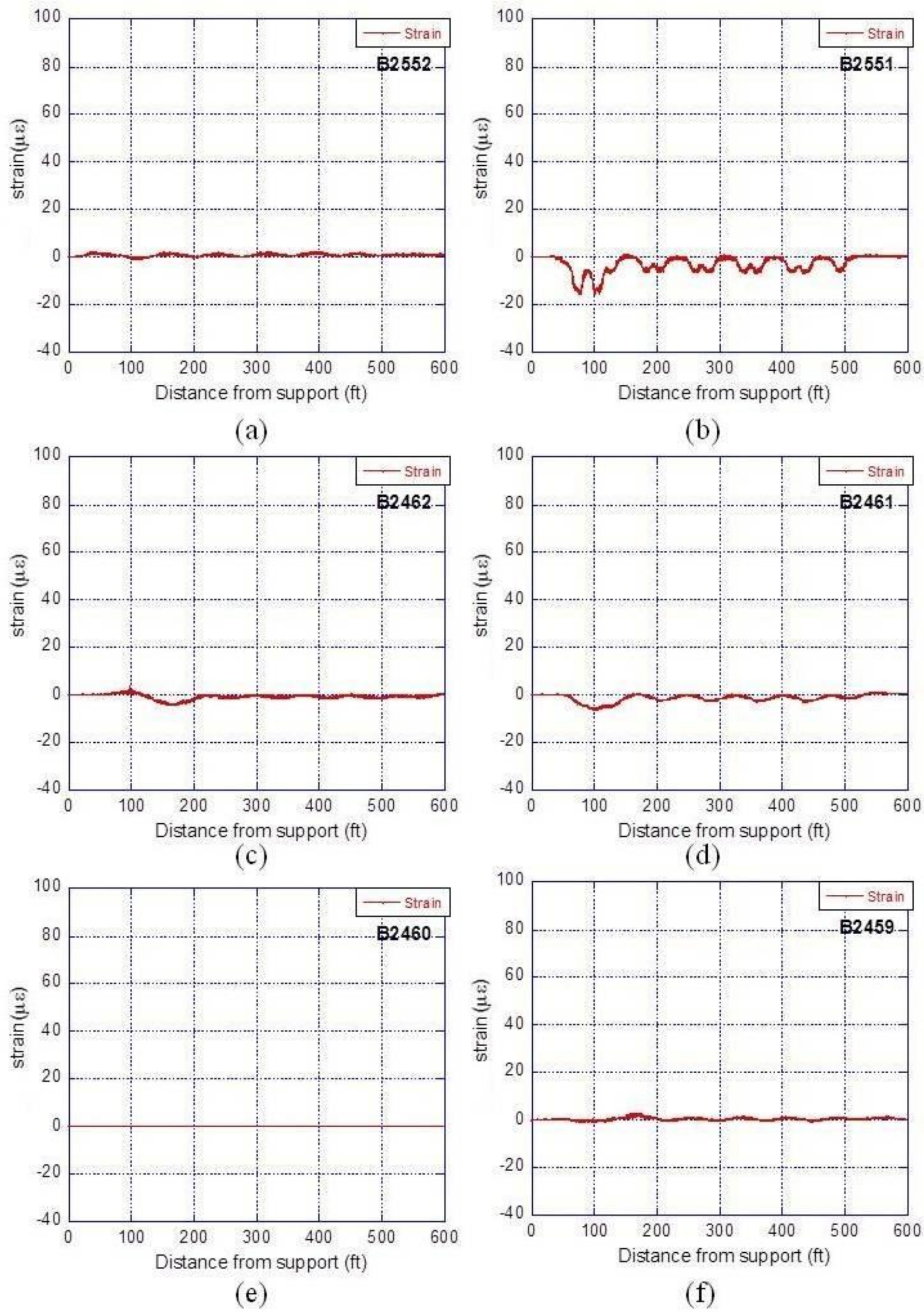


Figure 94.Strain data from run 1: *(a)* B2552,*(b)* B2551,*(c)* B2462,*(d)* B2461,*(e)* B2460, and *(f)* B2459

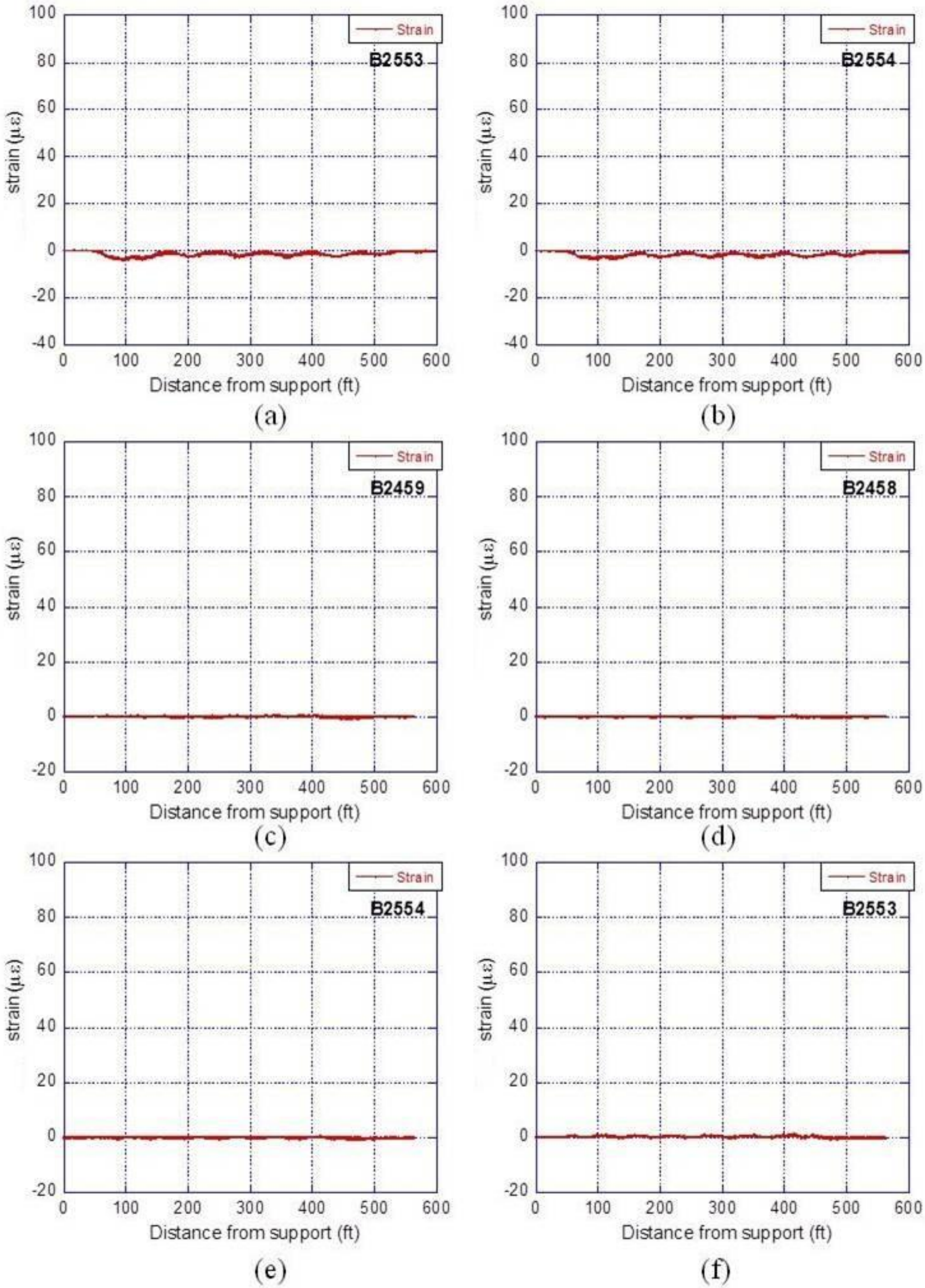


Figure 95.Strain data from run 1: *(a)* B2553,*(b)* B2554; and run 2: *(c)* B2459,*(d)* B2458,*(e)* B2554, and*(f)* B2553

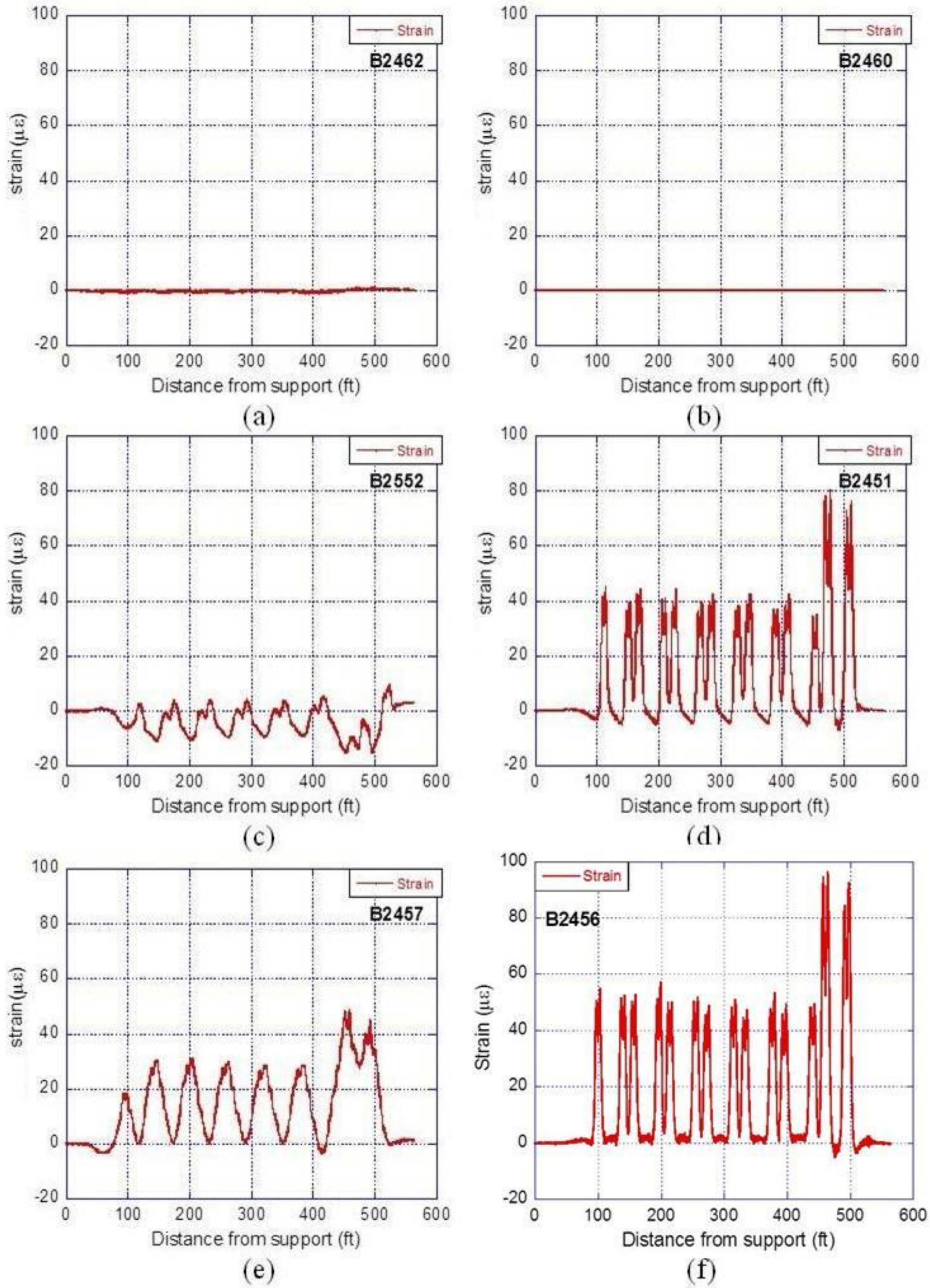


Figure 96.Strain data from run 2: *(a)* B2462,*(b)* B2460,*(c)* B2552,*(d)* B2451,*(e)* B2457, and *(f)* B2456

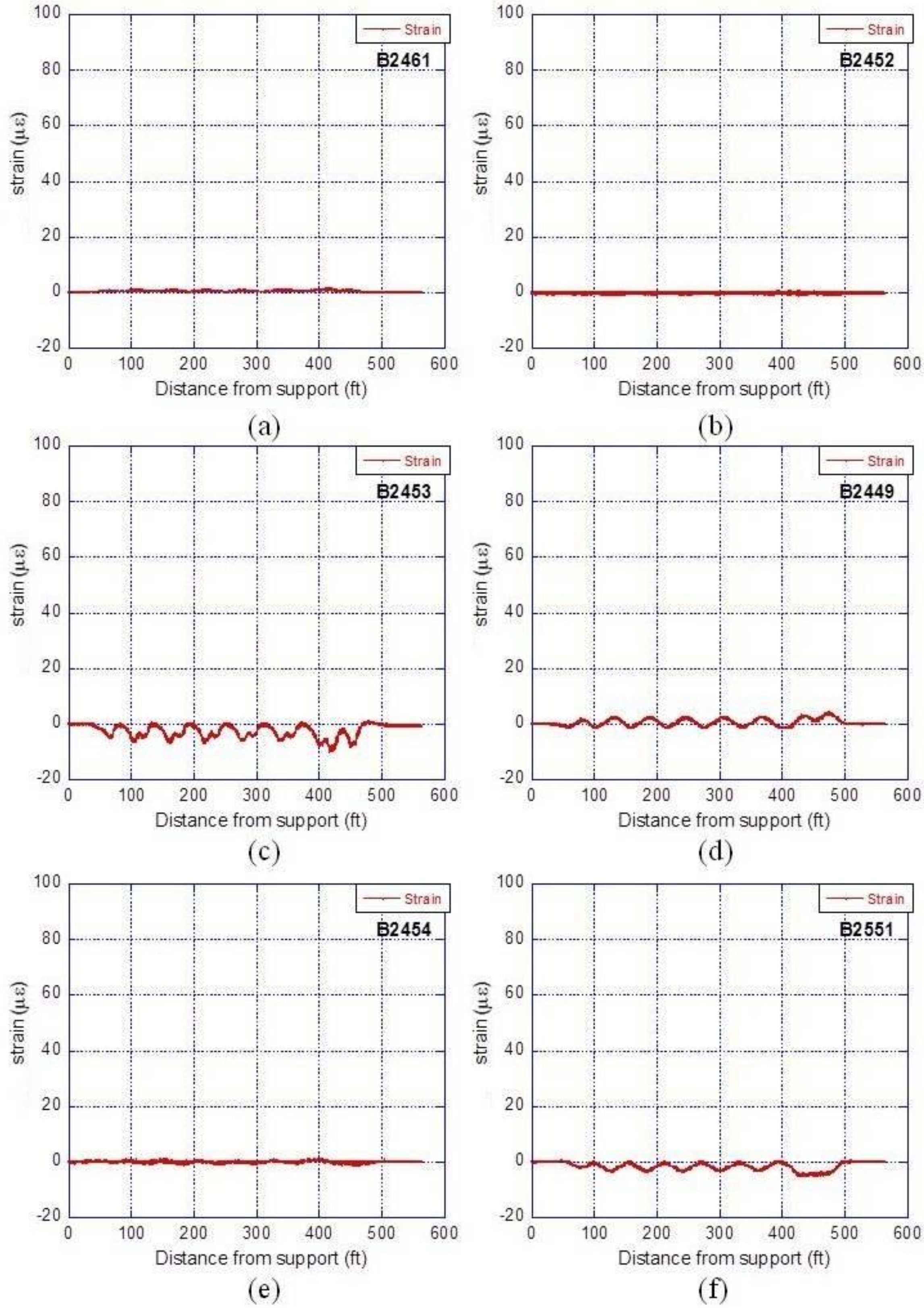


Figure 97.Strain data from run 2: *(a)* B2461,*(b)* B2452,*(c)* B2453,*(d)* B2449,*(e)* B2454, and *(f)* B2551

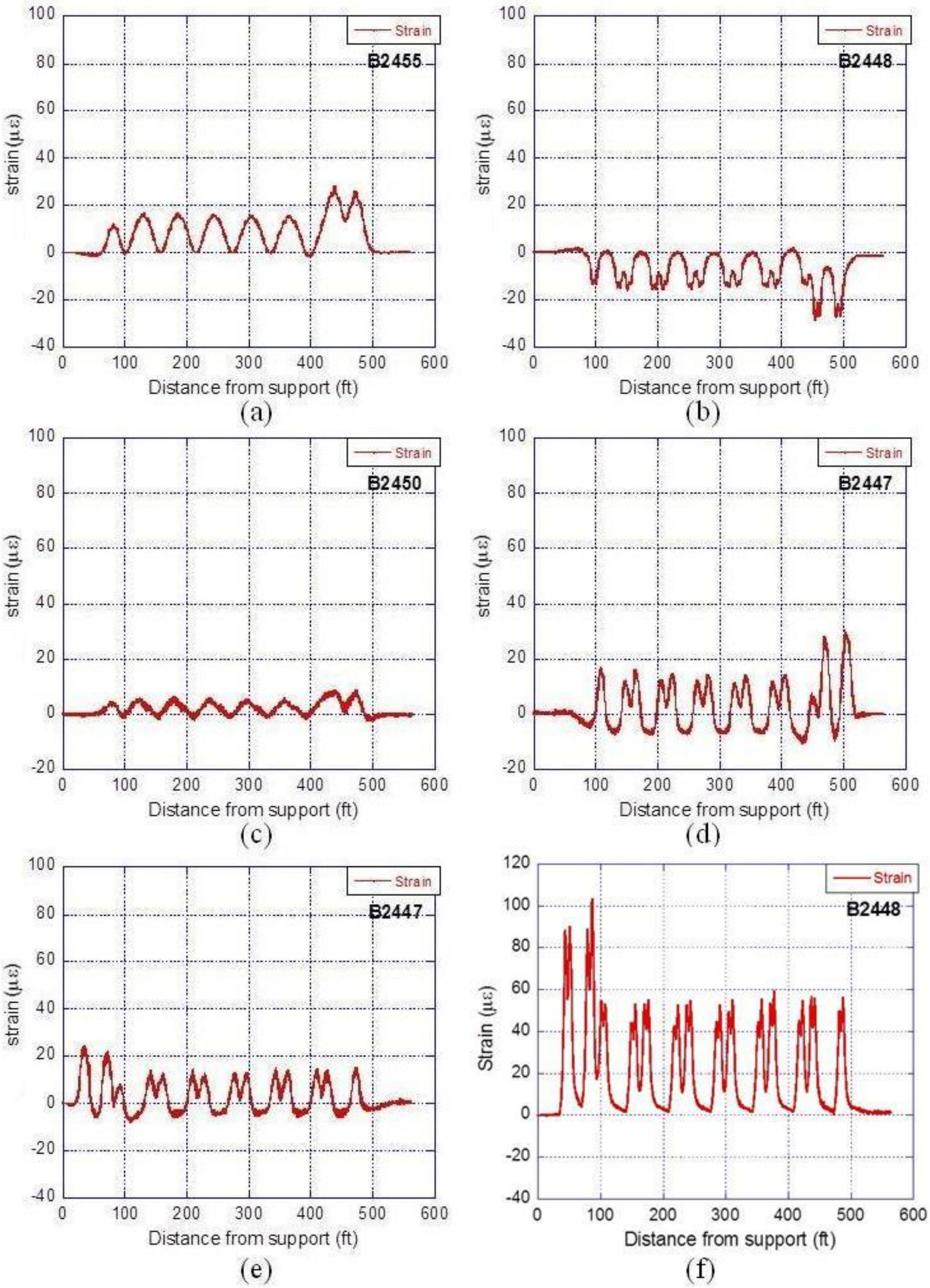


Figure 98.Strain data from run 2: *(a)* B2455,*(b)* B2448,*(c)* B2450,*(d)* B2447, case 3 *(e)* B2447, and *(f)* B2448

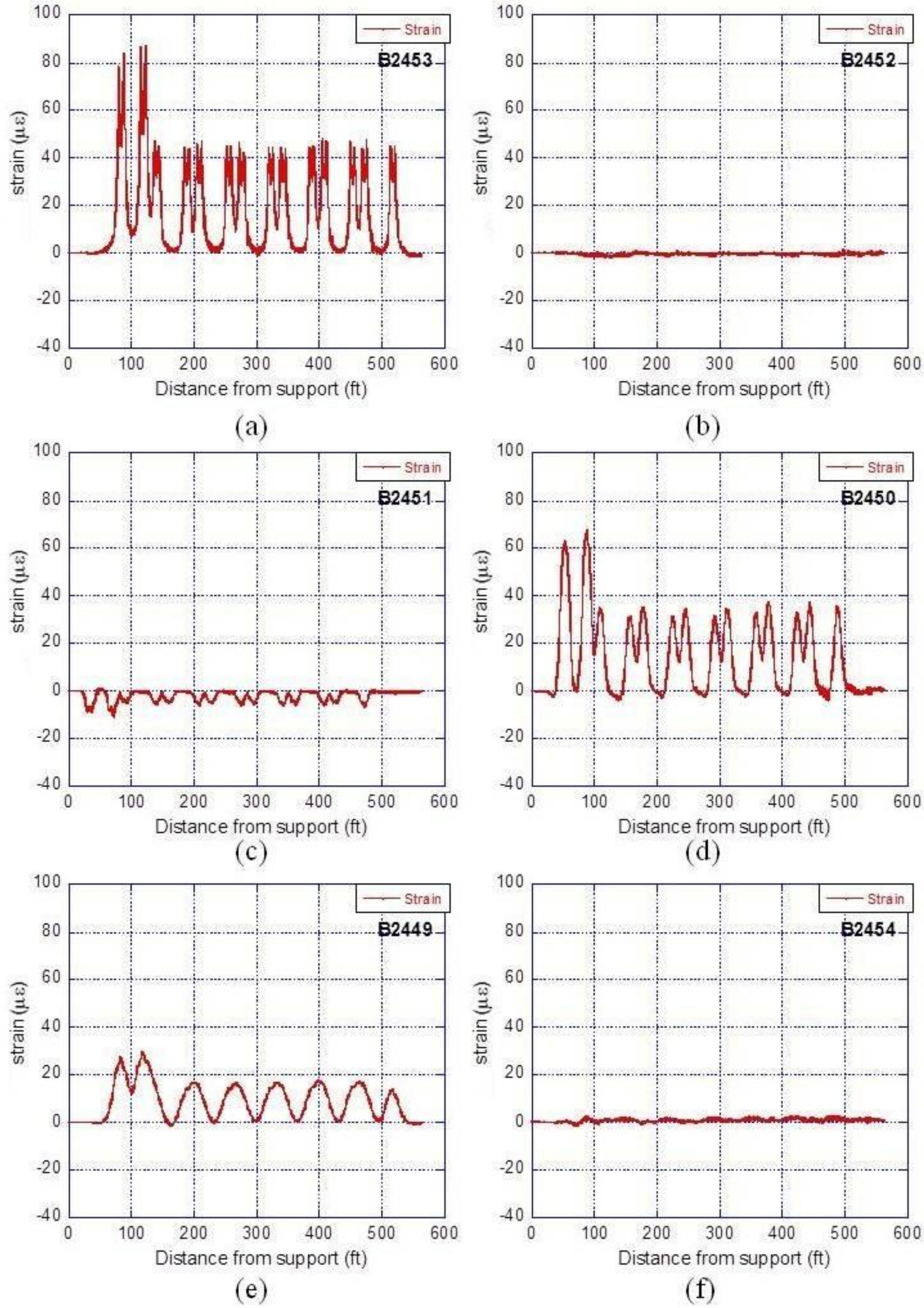


Figure 99.Strain data from run 3: *(a)* B2453,*(b)* B2452,*(c)* B2451,*(d)* B2450,*(e)* B2449, and *(f)* B2454

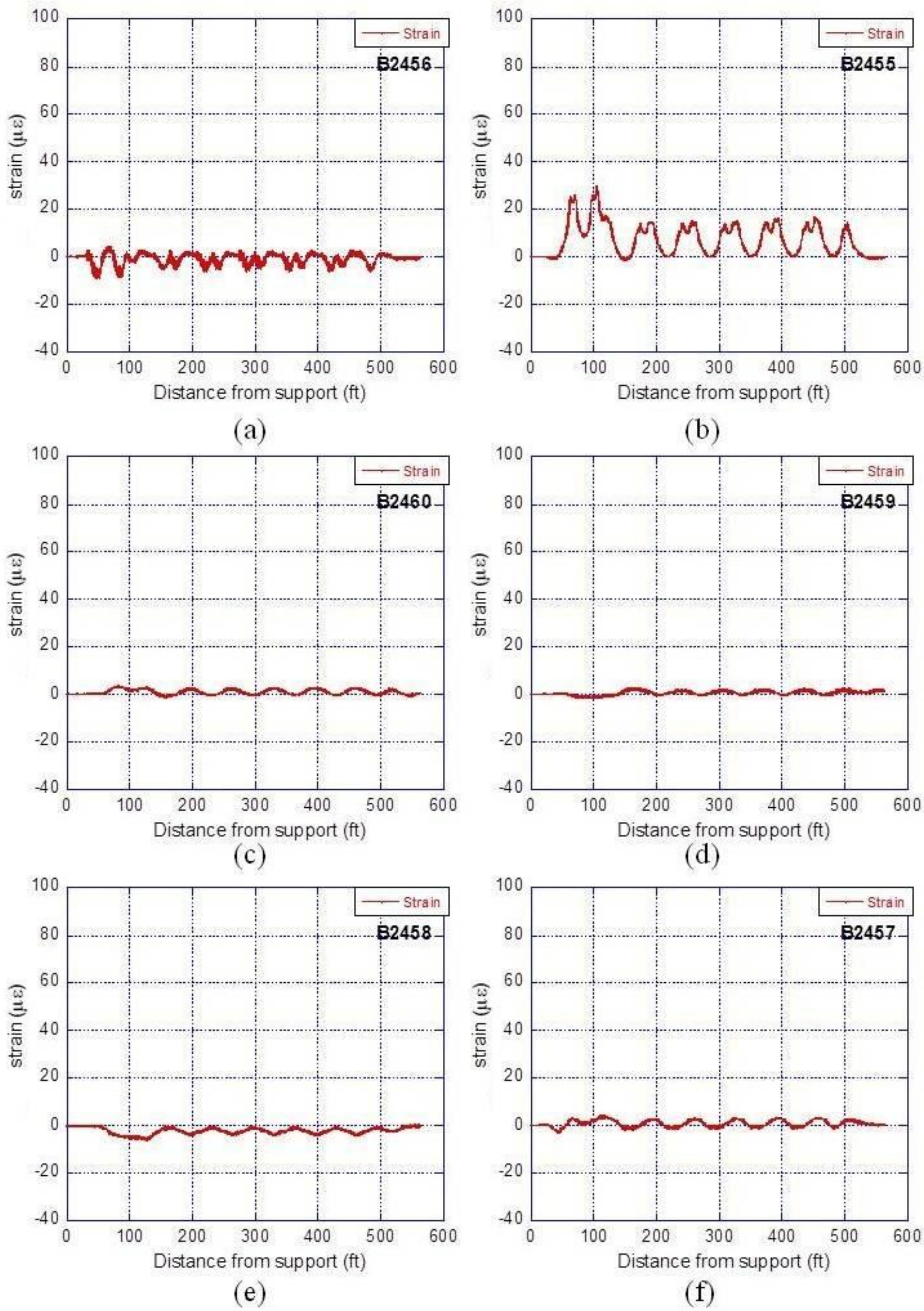


Figure 100.Strain data from run 3: *(a)* B2456,*(b)* B2455,*(c)* B2460,*(d)* B2459,*(e)* B2458, and *(f)* B2457

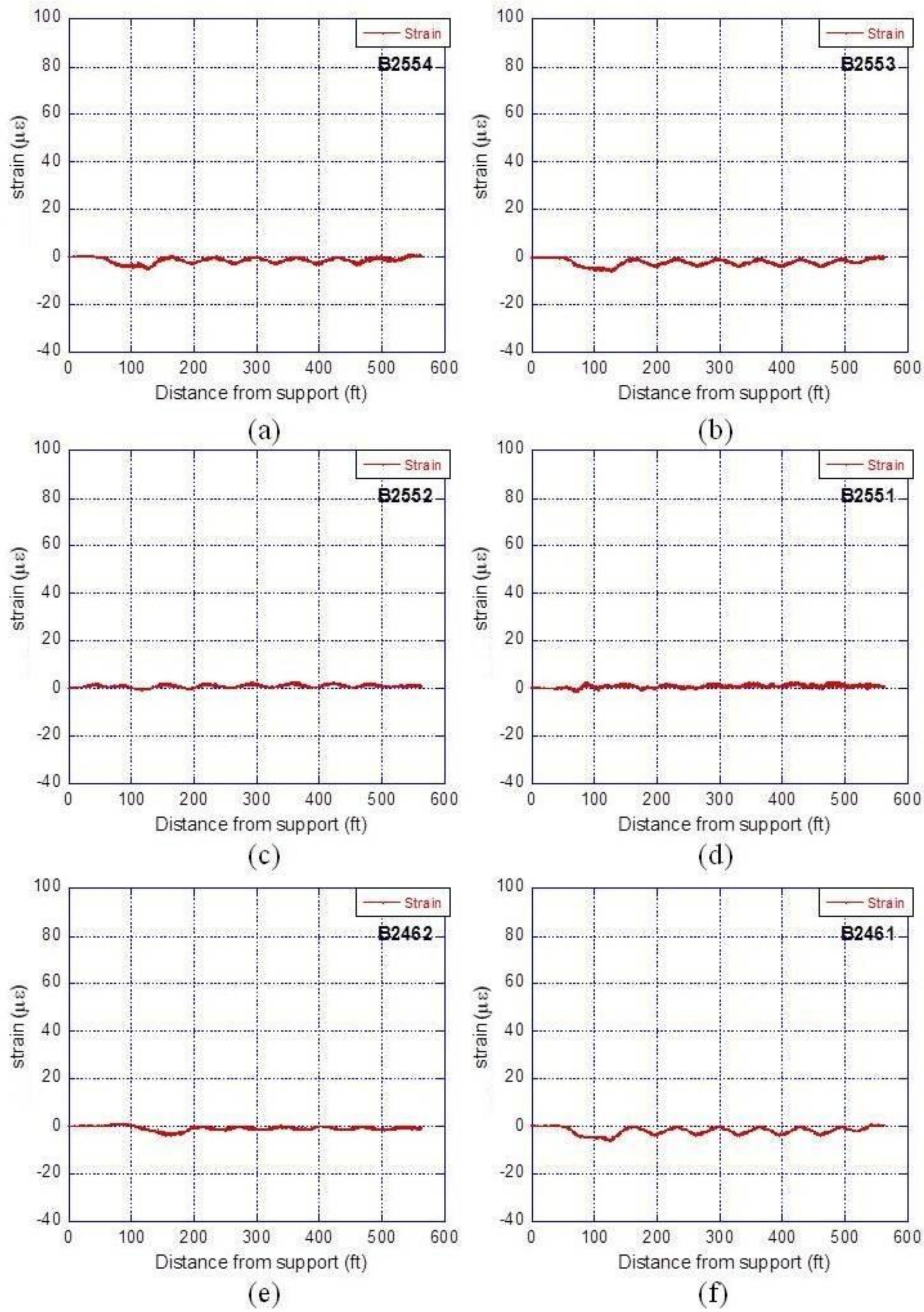


Figure 101.Strain data from run 3: *(a)* B2554,*(b)* B2553,*(c)* B2552,*(d)* B2551,*(e)* B2462, and *(f)* B2461

APPENDIX C: EXPERIMENTAL DATA COLLECTED FROM BERGEN COUNTY LINE MP 5.48 (HX DRAW) BRIDGE

Table 76 presents the information gathered from each experimental case, and Figure 102 through Figure 149 show the experimental data collected from field tests.

Table 76 - Information about the test trains for Bergen County Line MP 5.48 (HX Draw)

Run#	Time (a.m.)	Track	Direction	Type	Note
1	8:32	1	Westbound	Freight	
2	8:45	2		Freight	Static test
3	8:51	2		Freight	Static test
4	8:58	2		Freight	Static test
5	9:03	2	Westbound	Freight	10mph
6	9:10	2		Freight	Static test
7	9:15	1	Eastbound	Passenger	
8	9:18	2		Freight	Static test
9	9:20	1	Eastbound	Passenger	
10	9:25	2		Freight	Static test
11	9:30	2	Westbound	Freight	10mph
12	9:36	2	Eastbound	Freight	20mph
13	9:36	2	Westbound	Freight	20mph
14	9:40	2	Eastbound	Freight	25mph
15	9:45	2	Westbound	Freight	25mph
16	9:48	2	Eastbound	Freight	10mph
17	9:51	2	Westbound	Freight	10mph
18	9:54	2	Eastbound	Freight	20mph
19	9:58	2	Westbound	Freight	20mph
20	10:00	2	Eastbound	Freight	25mph
21	10:04	2	Westbound	Freight	25mph
22	10:14	1		Passenger	
23	10:21	1		Freight	Static test
24	10:28	1		Freight	Static test
25	10:29	1		Freight	Static test
26	10:30	1		Freight	No train passed
27	10:31	1		Freight	Static test
28	10:44	1	Westbound	Passenger	
29	10:49	1	Westbound	Freight	10mph
30	10:53	1	Eastbound	Freight	10mph
31	10:56	1	Westbound	Freight	20mph
32	11:00	1	Eastbound	Freight	20mph
33	11:06	1	Westbound	Freight	25mph
34	11:09	1	Eastbound	Freight	25mph

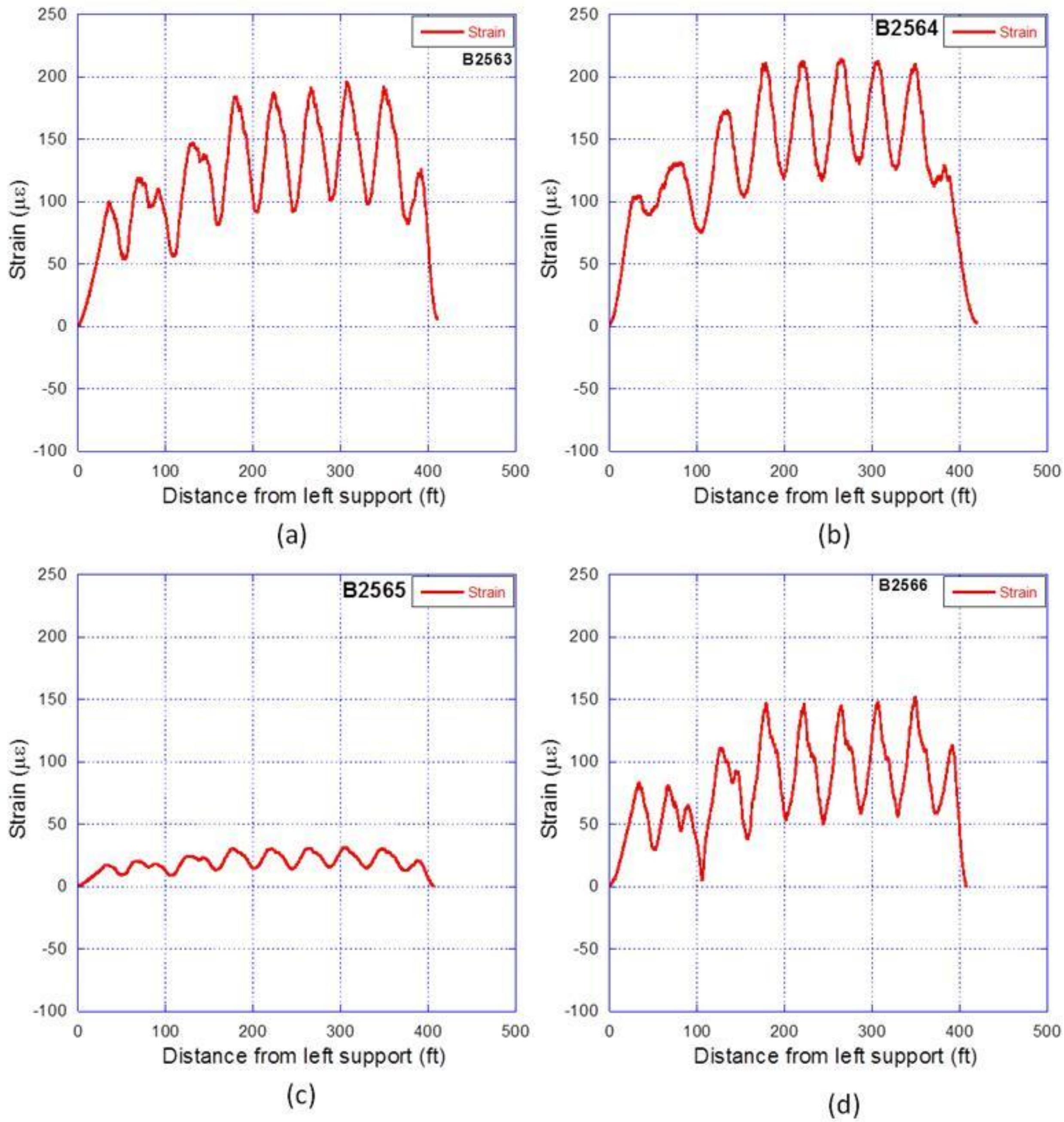


Figure 102: Strain data during Test Run #5 at strain transducer station *(a)* B2563, *(b)* B2564, *(c)* B2565, and *(d)*B2566

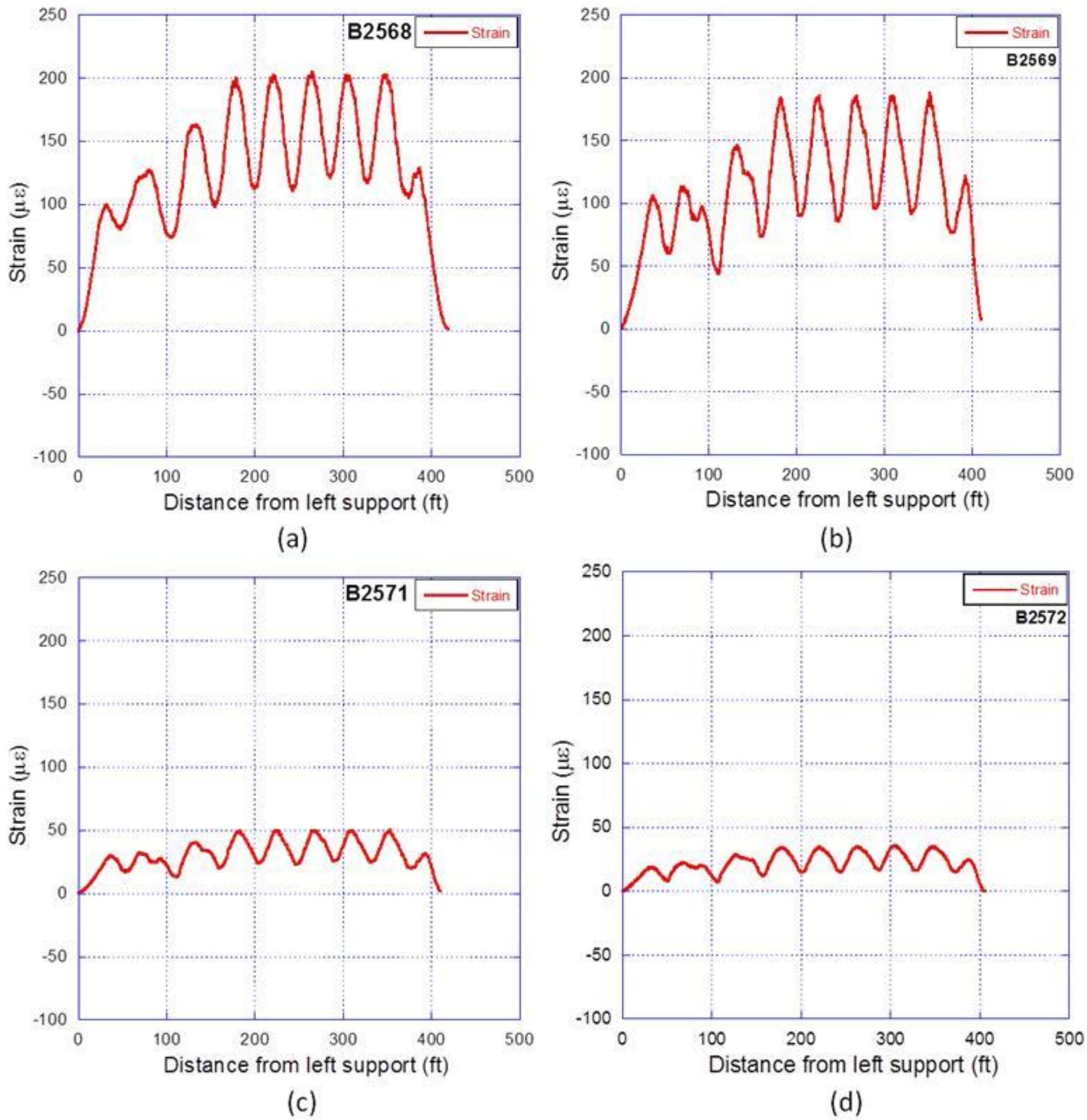


Figure 103:Strain data during Test Run #5 at strain transducer station*(a)* B2568, *(b)* B2569, *(c)* B2571, and *(d)* B2572

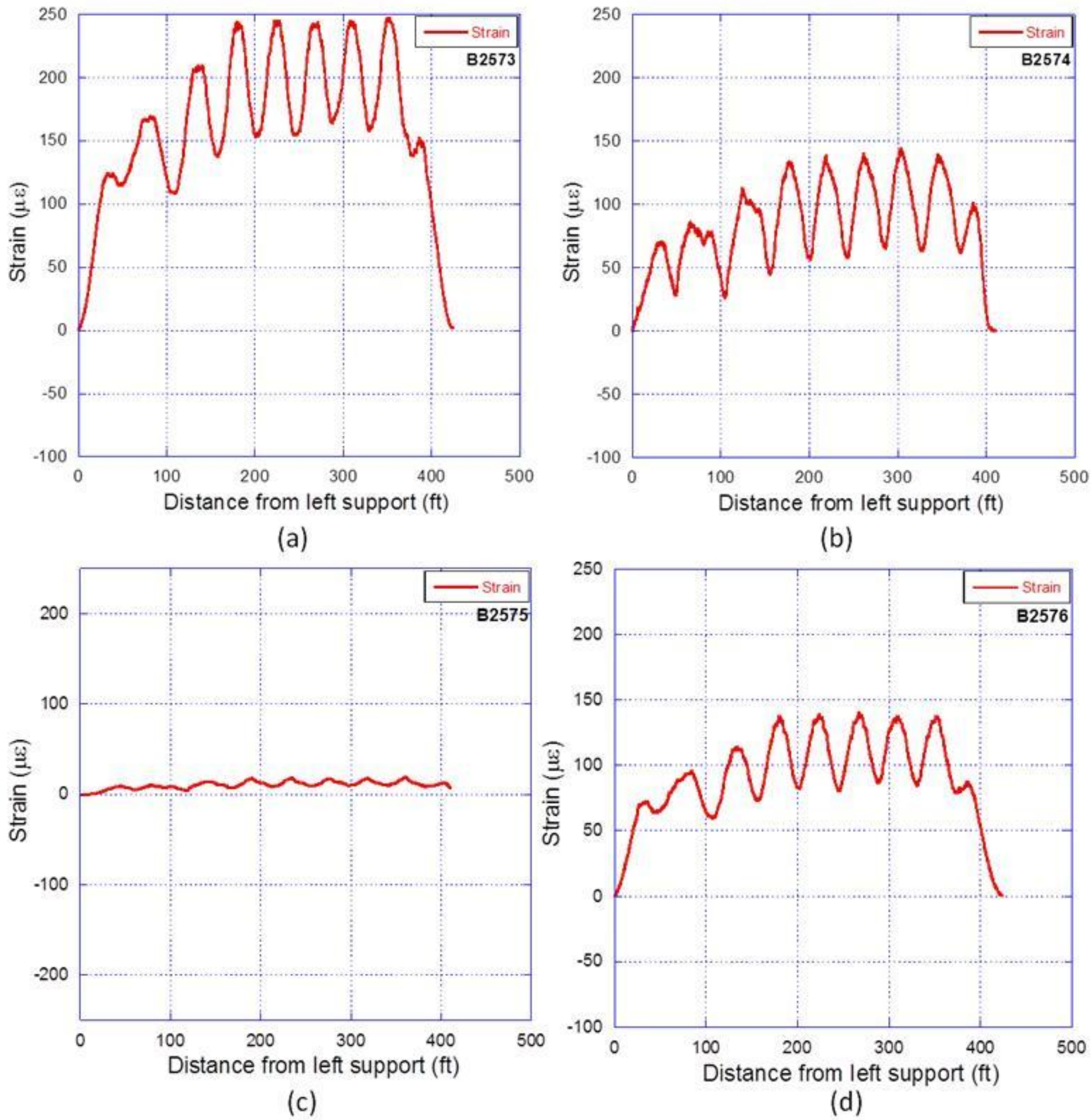


Figure 104: Strain data during Test Run #5 at strain transducer station *(a)* B2573, *(b)* B2574,*(c)* B2575, and *(d)* B2576

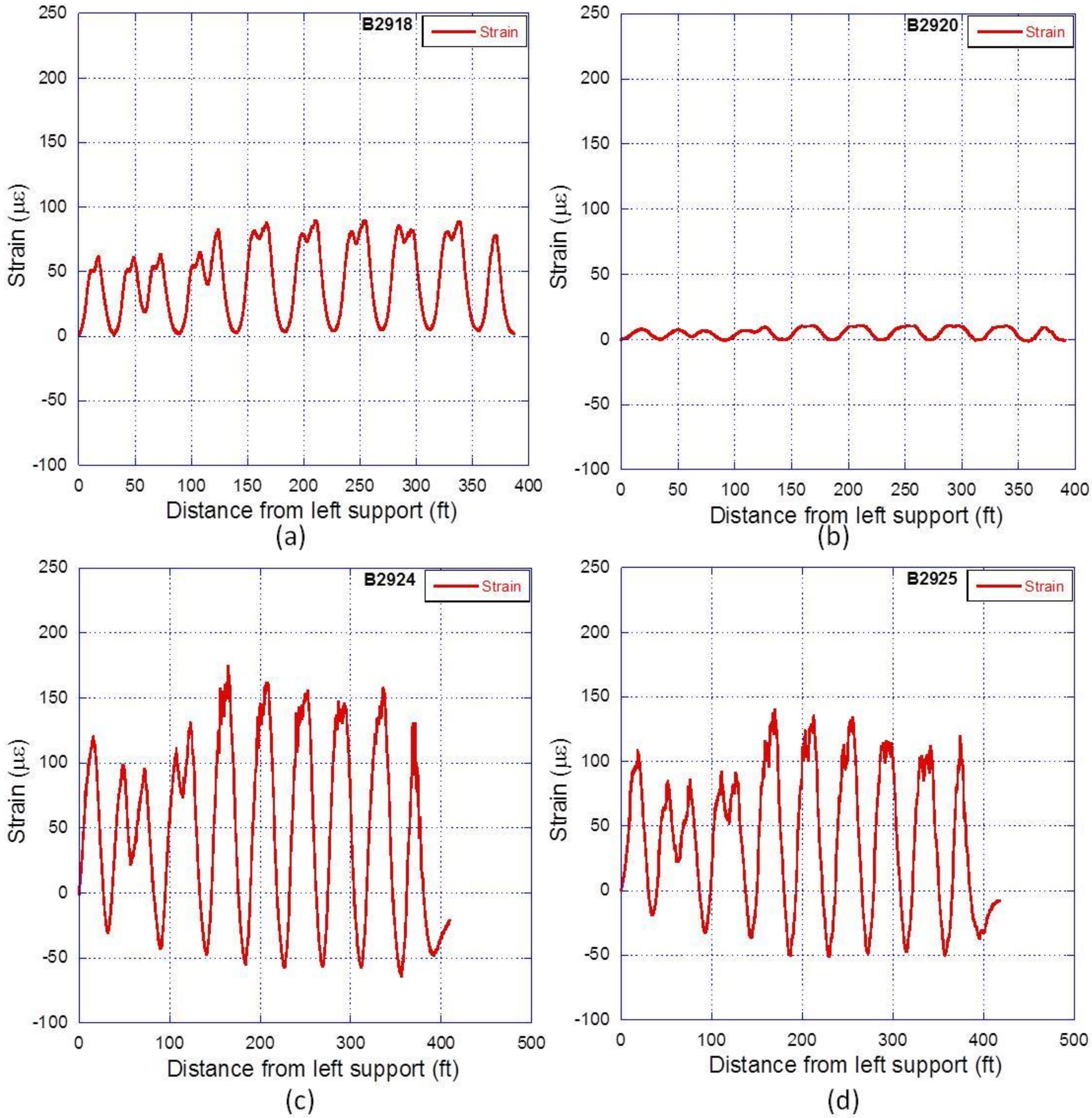


Figure 105: Strain data during Test Run #5 at strain transducer station *(a)* B2918, *(b)* B2920,*(c)* B2924, and*(d)* B2925

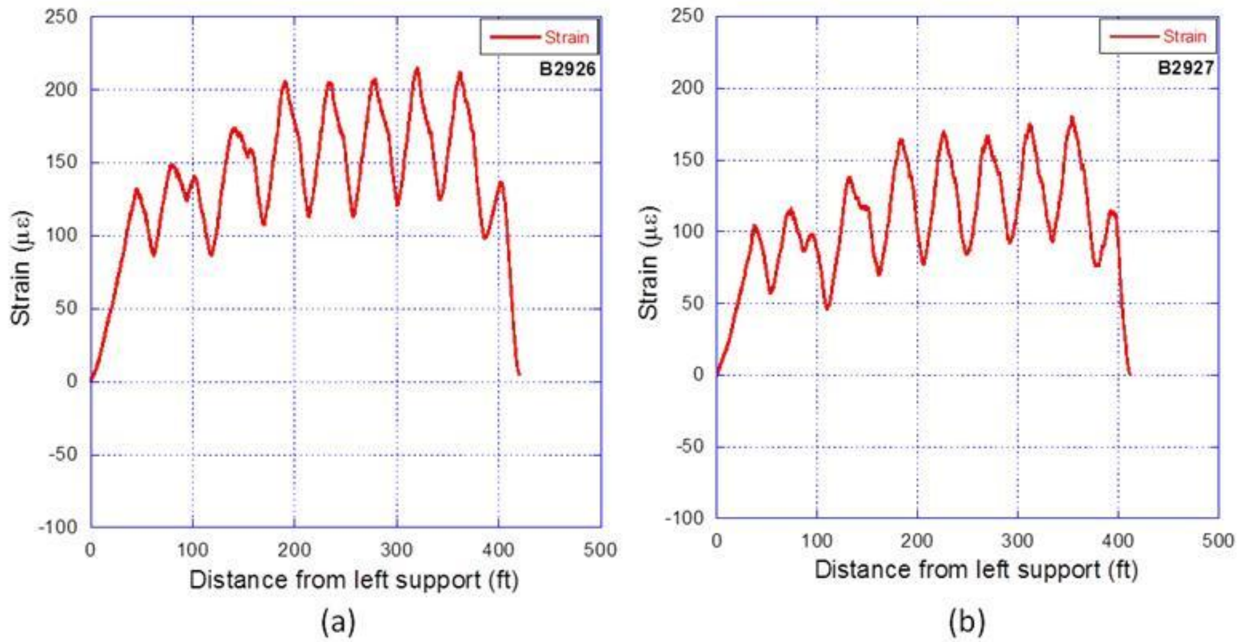


Figure 106: Strain data during Test Run #5 at strain transducer station *(a)* B2926 and *(b)*B2927

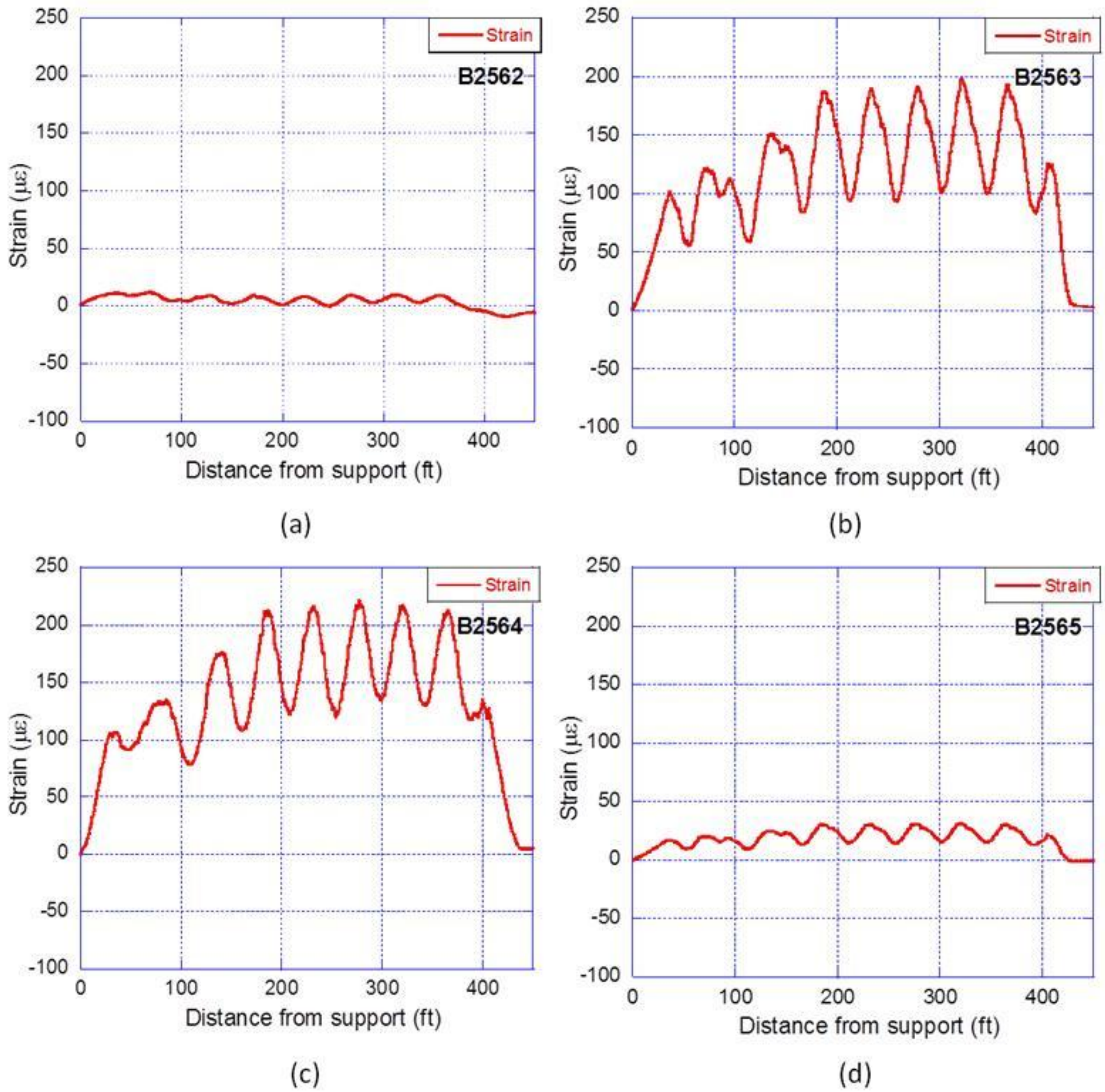


Figure 107: Strain data during Test Run #11 at strain transducer station *(a)* B2562, *(b)* B2563, *(c)* B2564, and *(d)* B2565

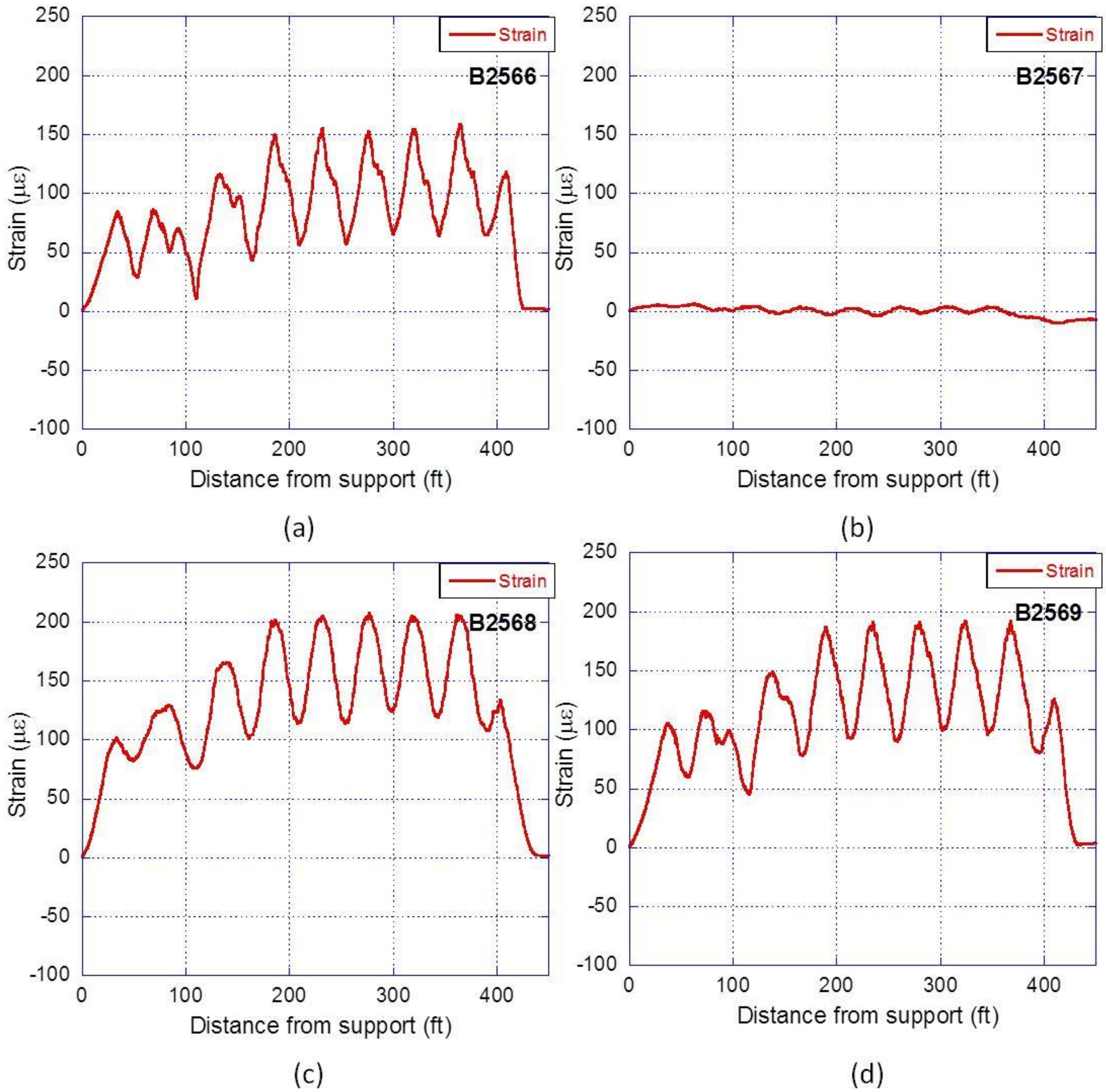


Figure 108: Strain data during Test Run #11 at strain transducer station *(a)* B2566, *(b)* B2567, *(c)* B2568, and *(d)* B2569

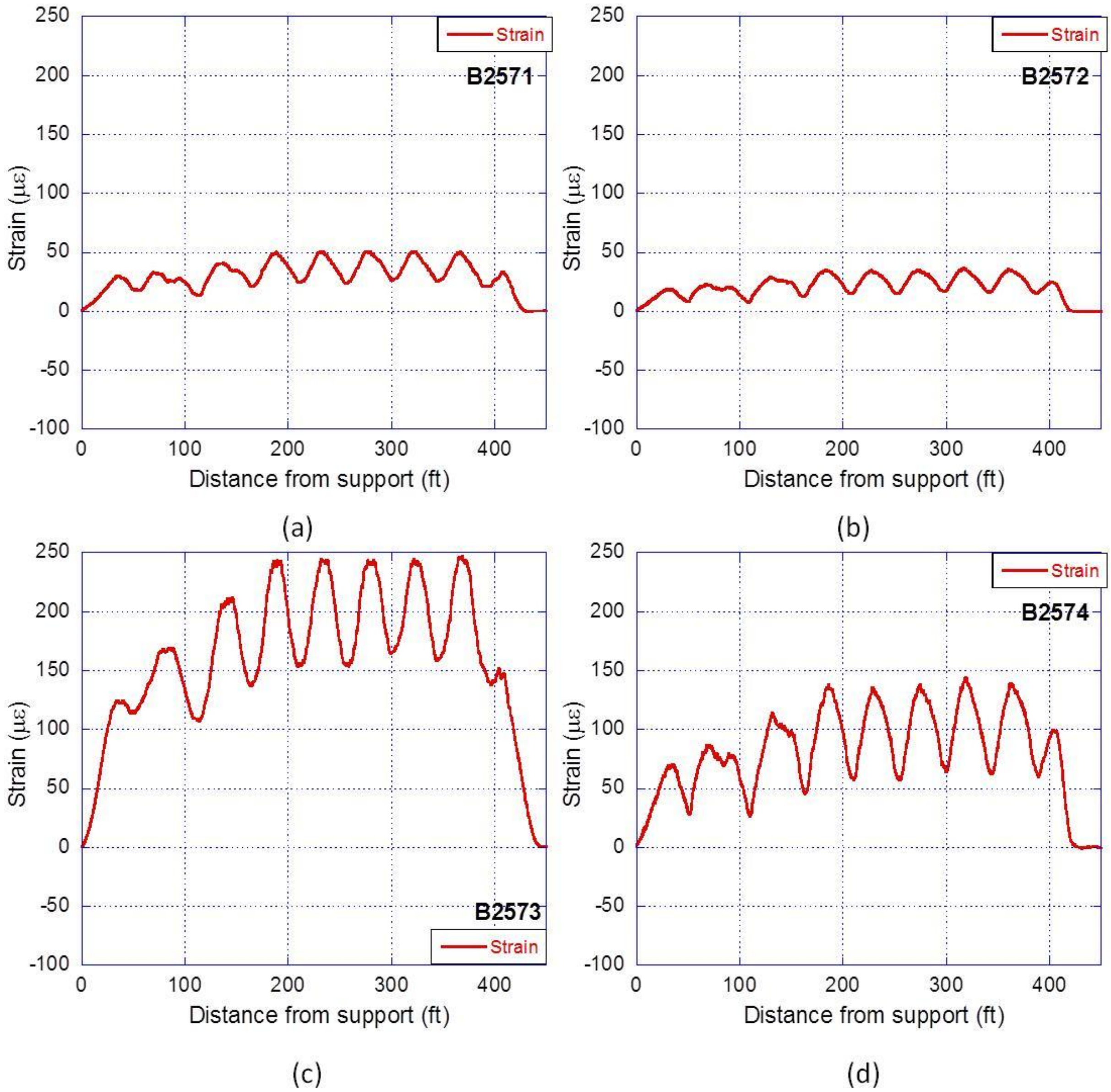


Figure 109: Strain data during Test Run #11 at strain transducer station *(a)* B2571, *(b)* B2572, *(c)* B2573, and*(d)* B2574

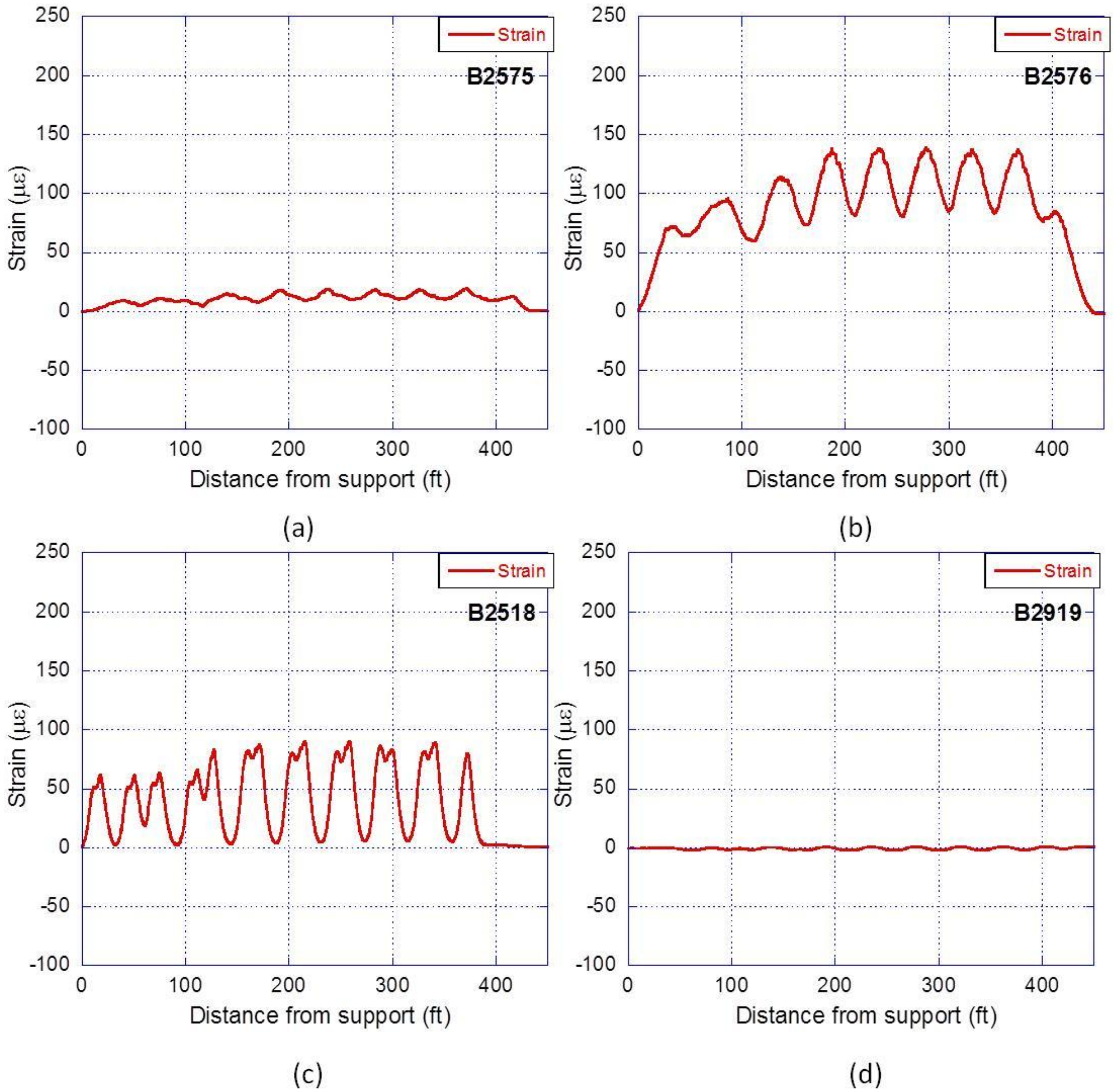


Figure 110: Strain data during Test Run #11 at strain transducer station *(a)* B2575, *(b)* B2576, *(c)* B2918, and *(d)* B2919

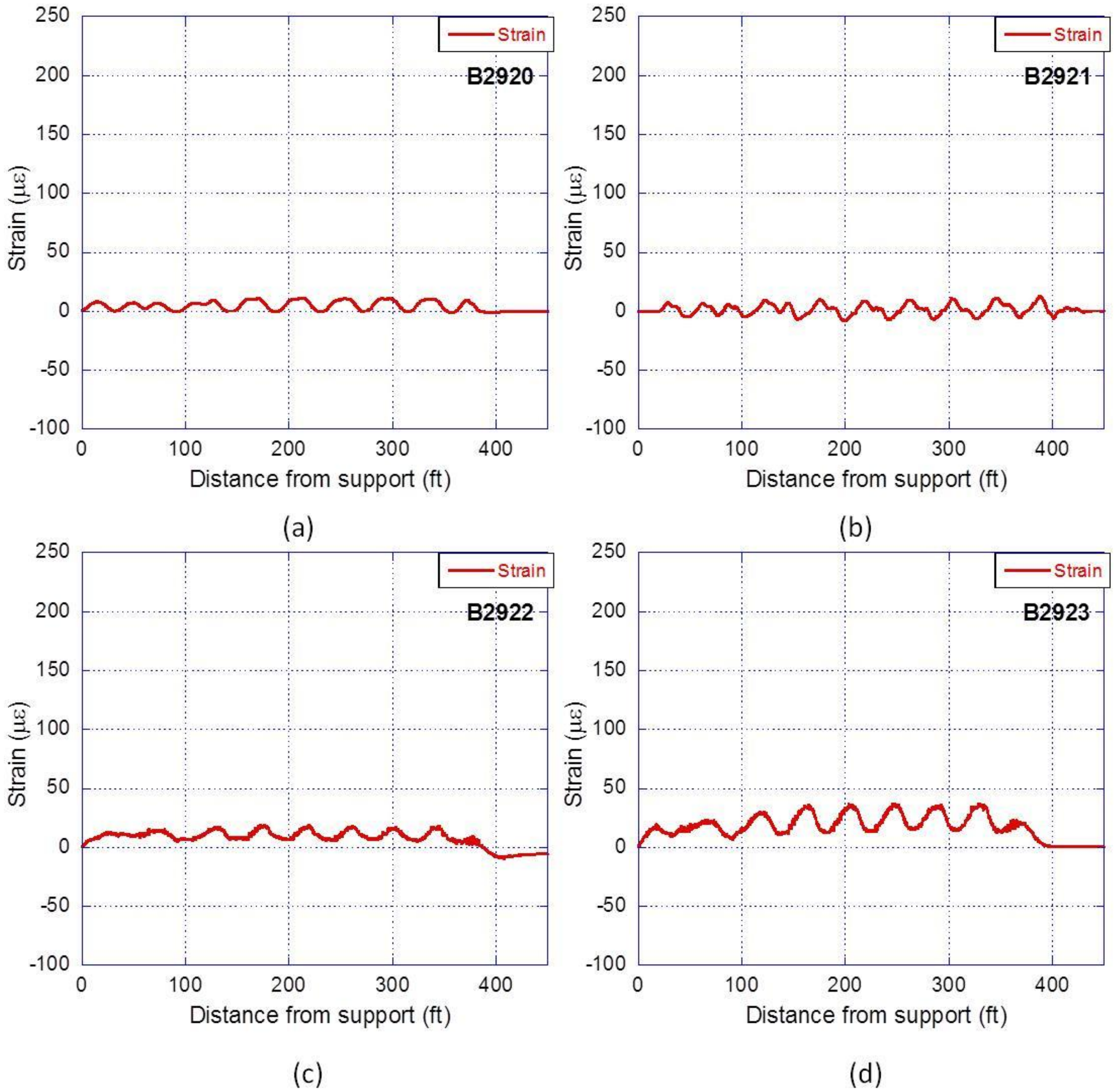


Figure 111: Strain data during Test Run #11 at strain transducer station *(a)* B2920, *(b)* B2921, (c) B922, and *(d)* B2923

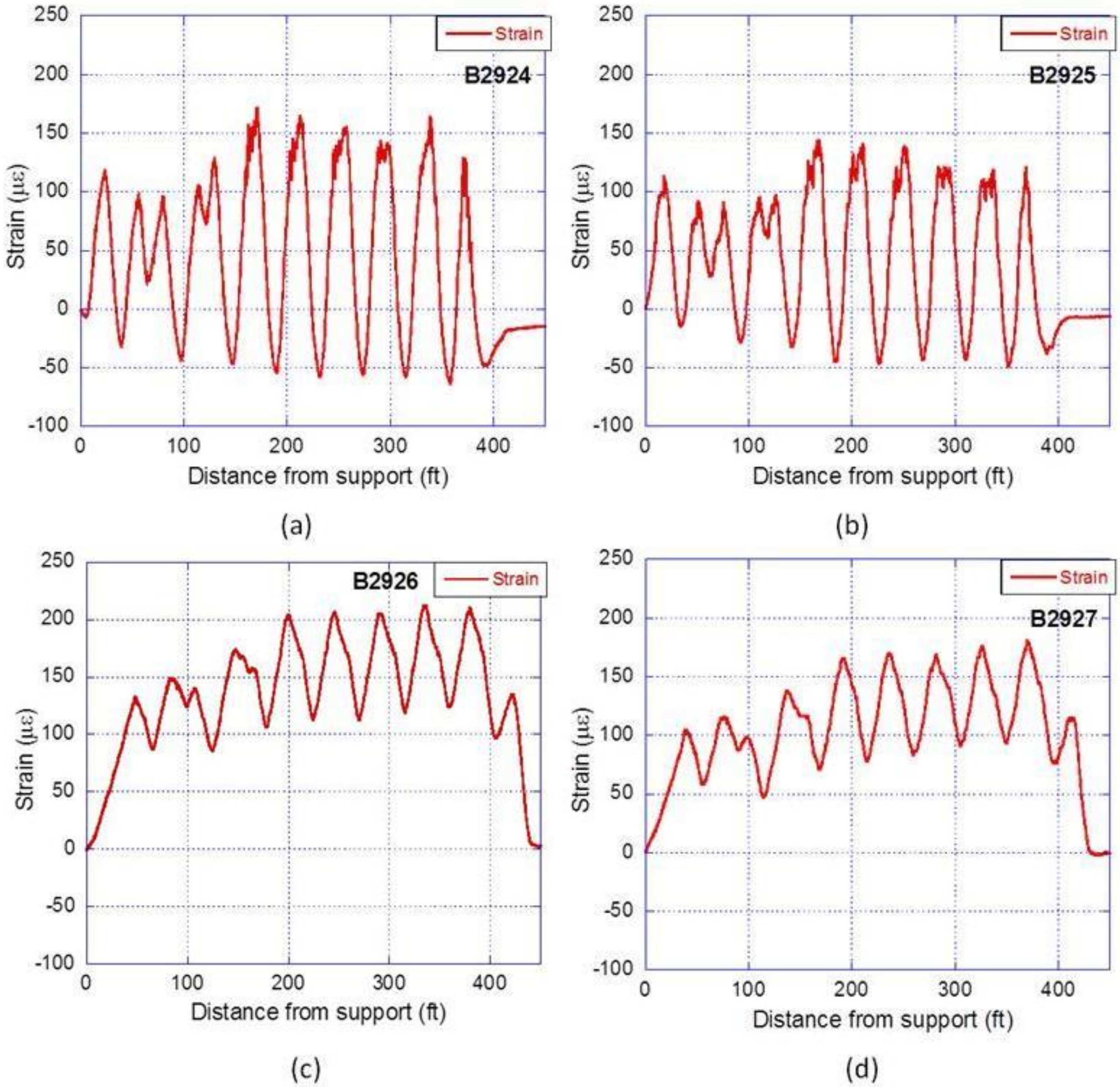


Figure 112: Strain data during Test Run #11 at strain transducer station *(a)* B2924, *(b)* B2925, *(c)* B2926, and *(d)* B2927

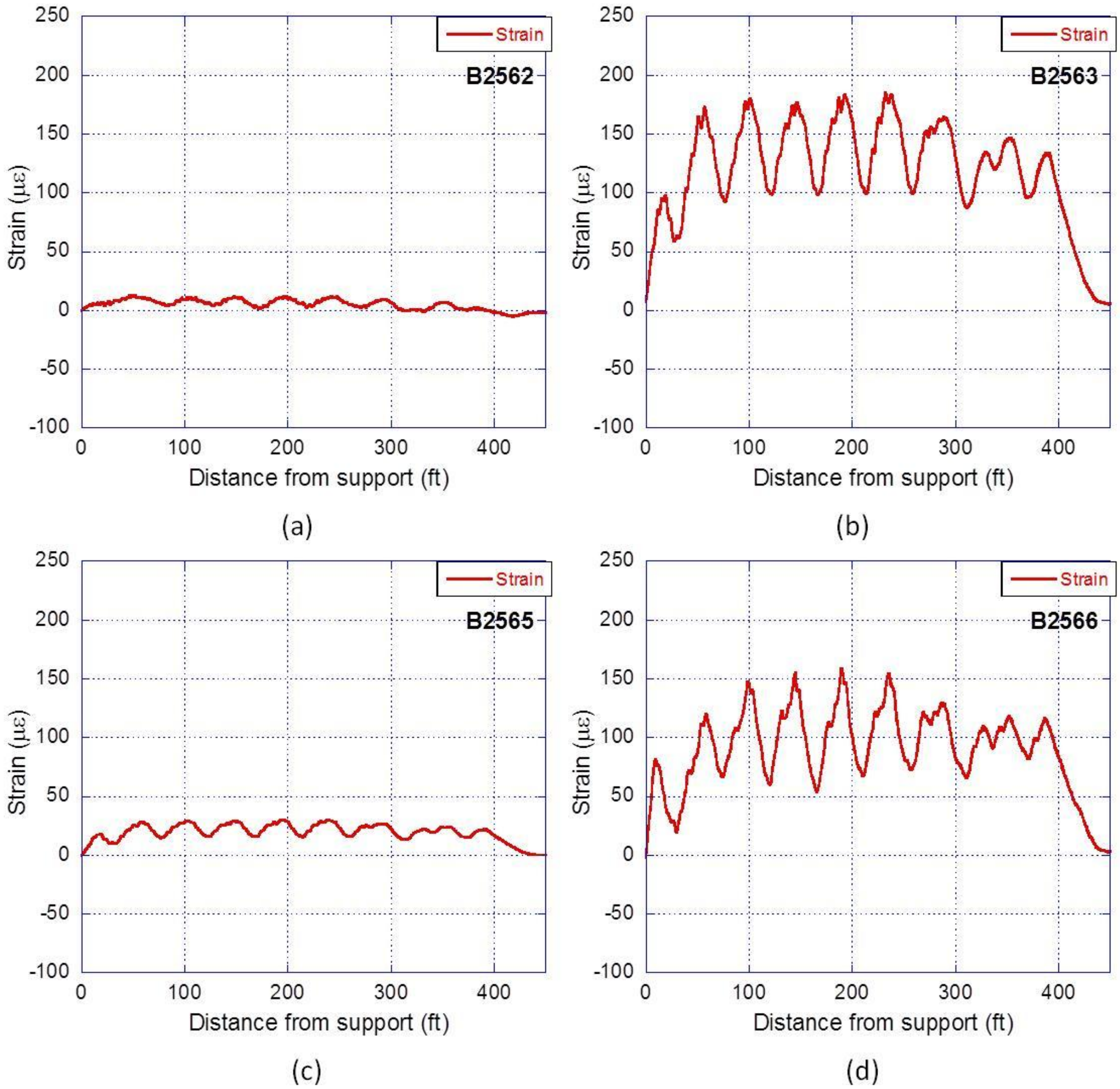


Figure 113: Strain data during Test Run #12 at strain transducer station *(a)* B2562, *(b)* B2563, *(c)* B2565, and *(d)* B2566

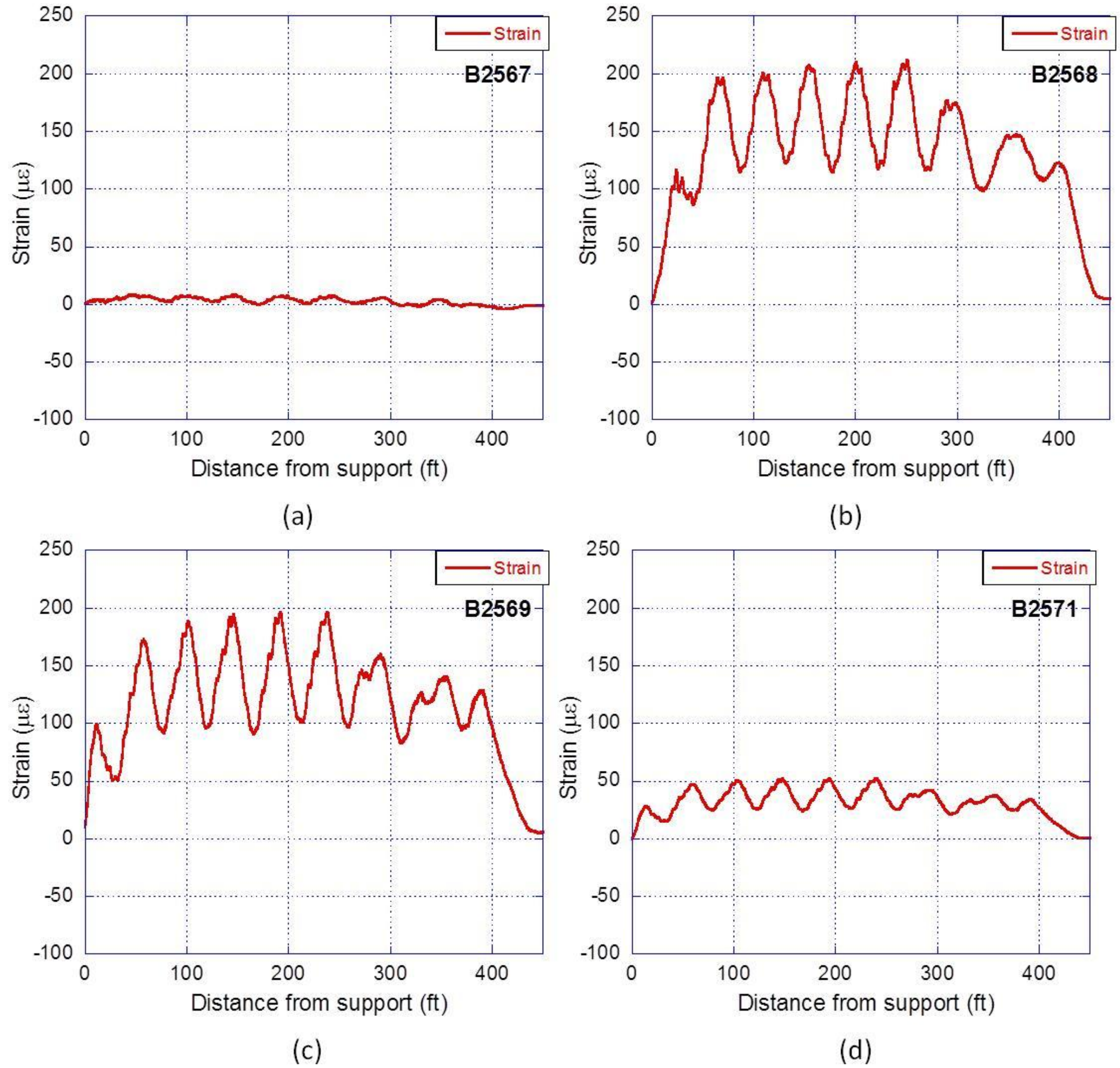


Figure 114: Strain data during Test Run #12 at strain transducer station *(a)* B2567, *(b)* B2568, *(c)* B2269, and *(d)* B2271.

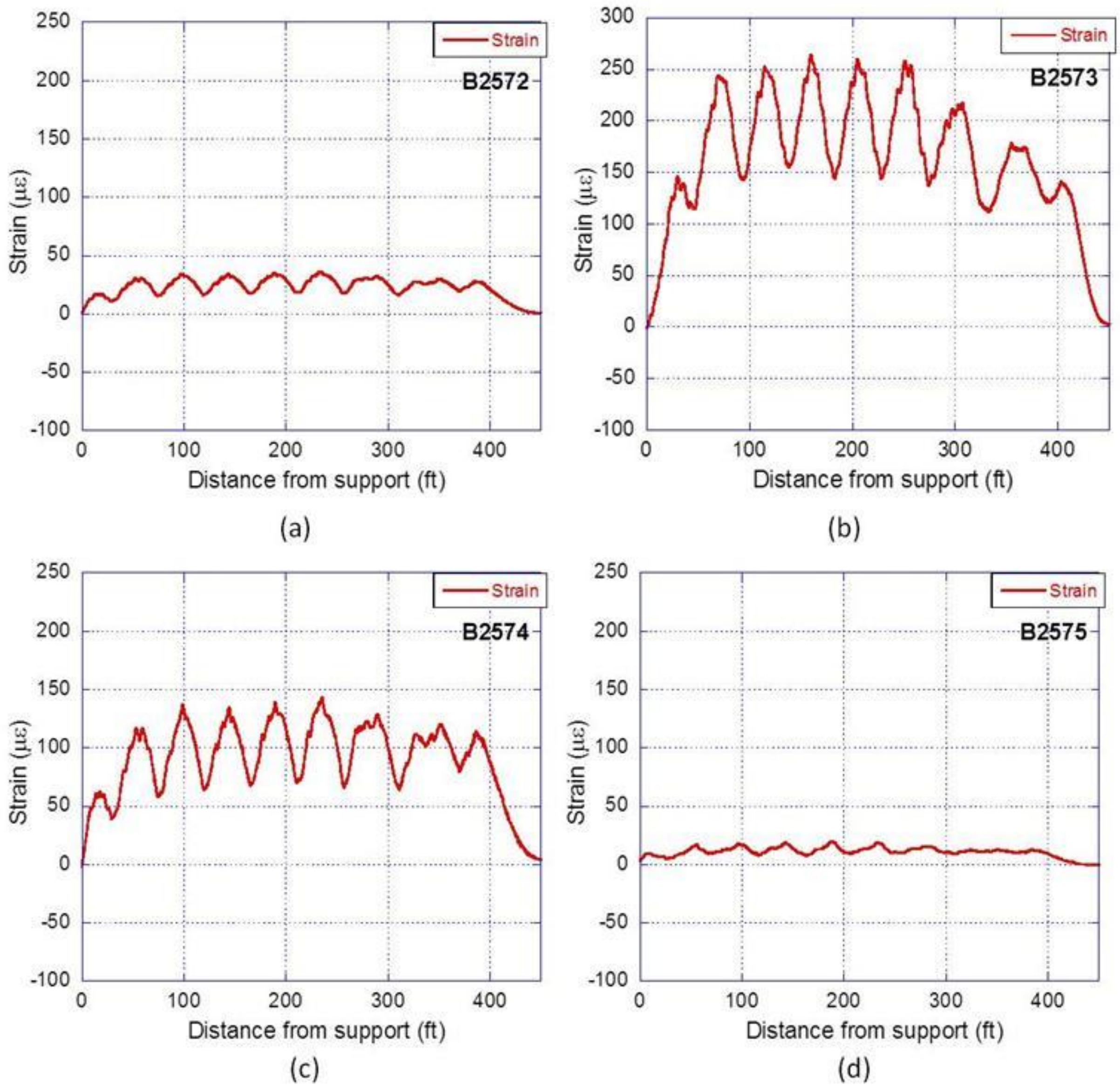


Figure 115: Strain data during Test Run #12 at strain transducer station *(a)* B2572, *(b)* B2573, *(c)* B2274, and *(d)* B2275

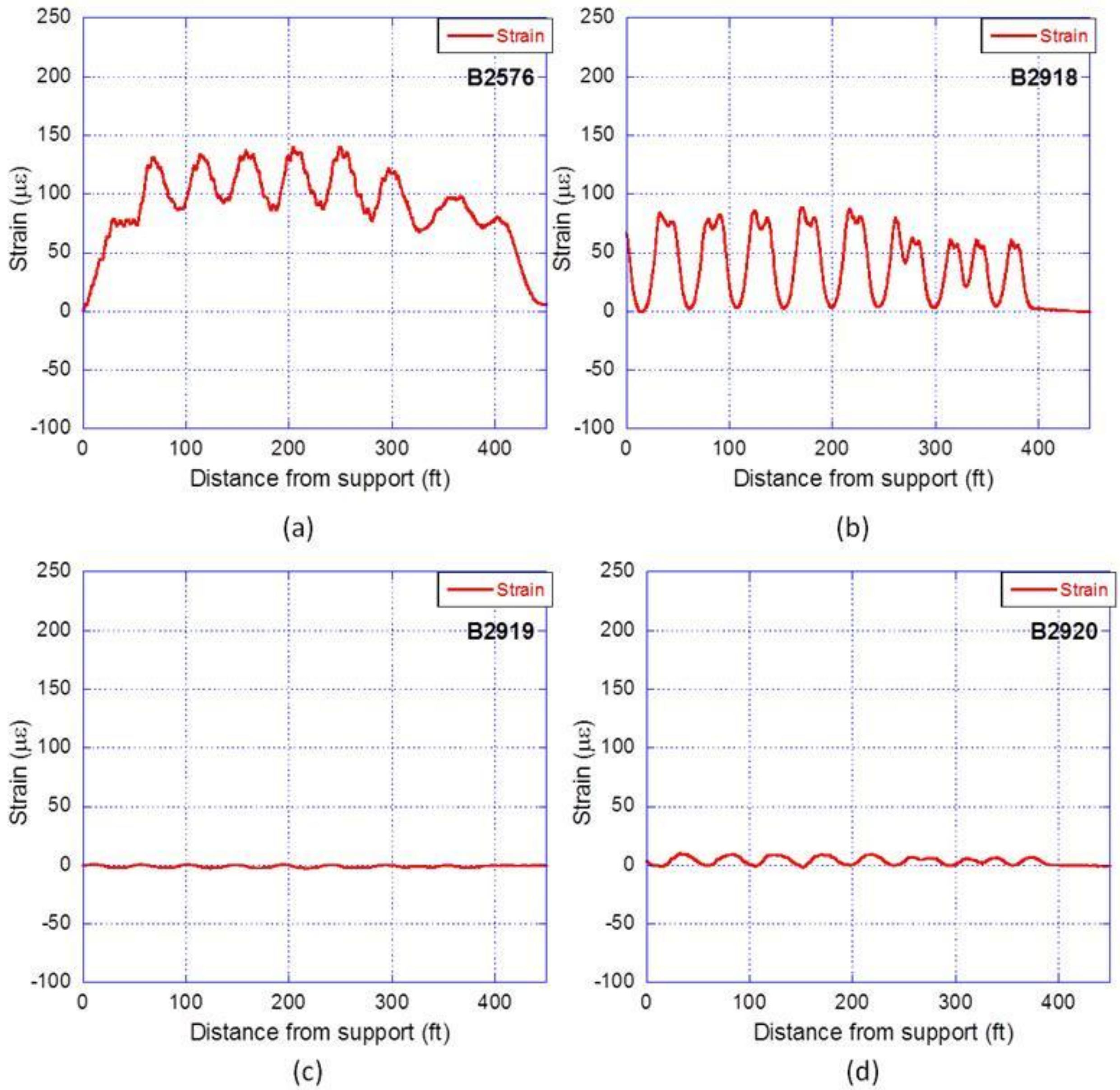


Figure 116: Strain data during Test Run #12 at strain transducer station *(a)* B2576, *(b)* B2918, *(c)* B2919, and *(d)* B2920

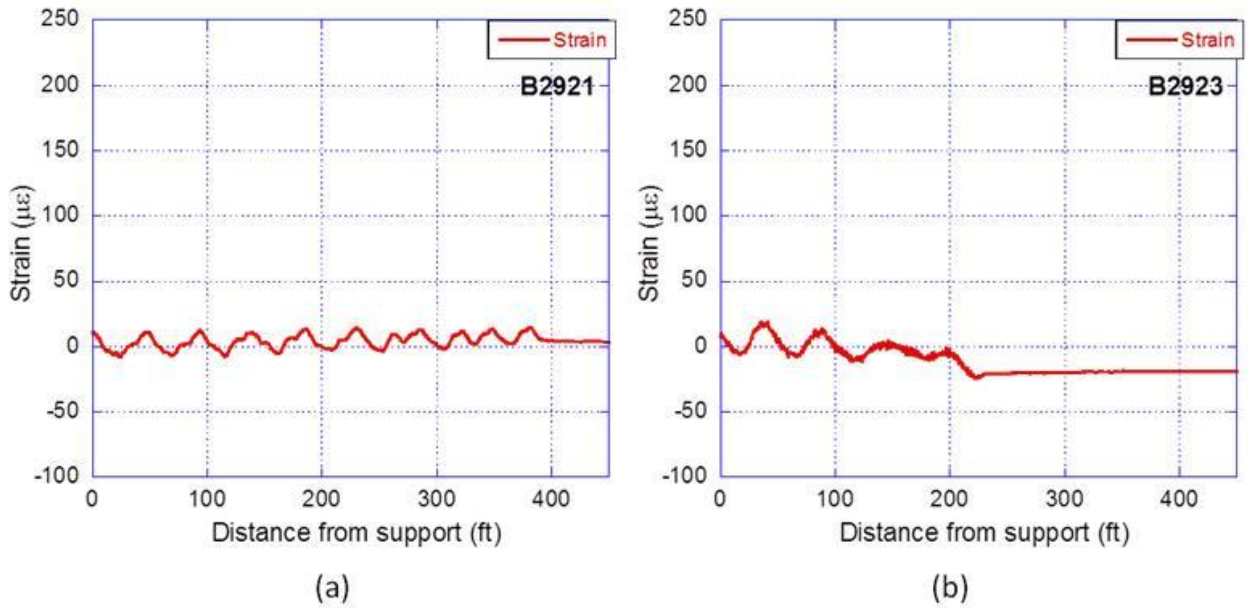


Figure 117: Strain data during Test Run #12 at strain transducer station *(a)* B2921and *(b)* B2923

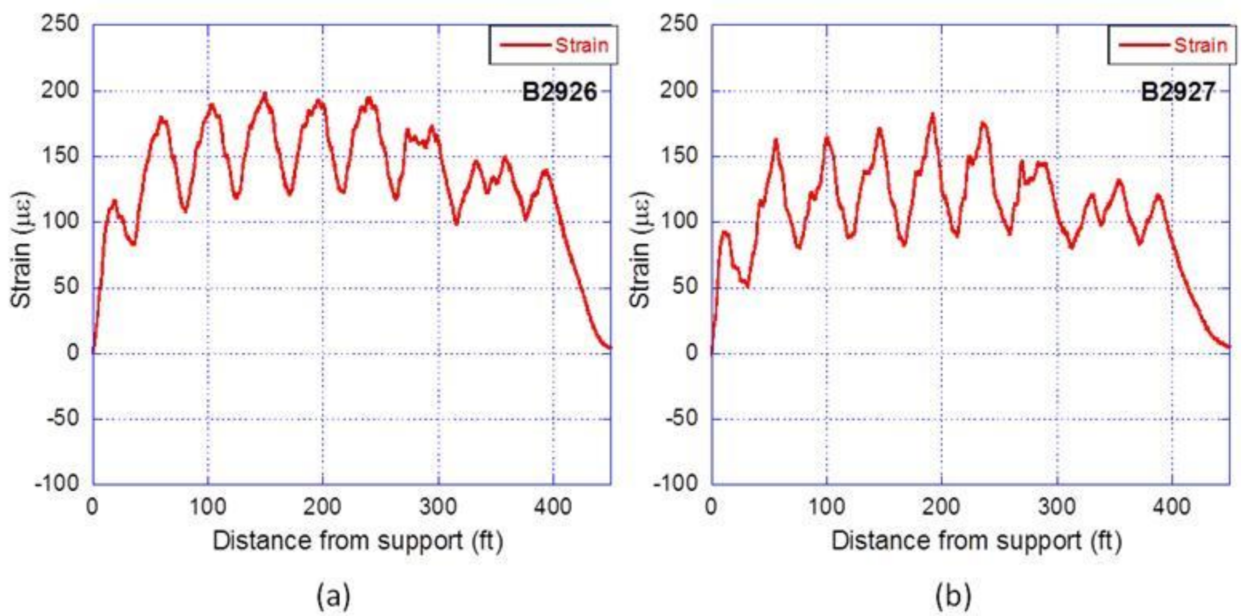


Figure 118: Strain data during Test Run #12 at strain transducer station *(a)* B2926and *(b)* B2927

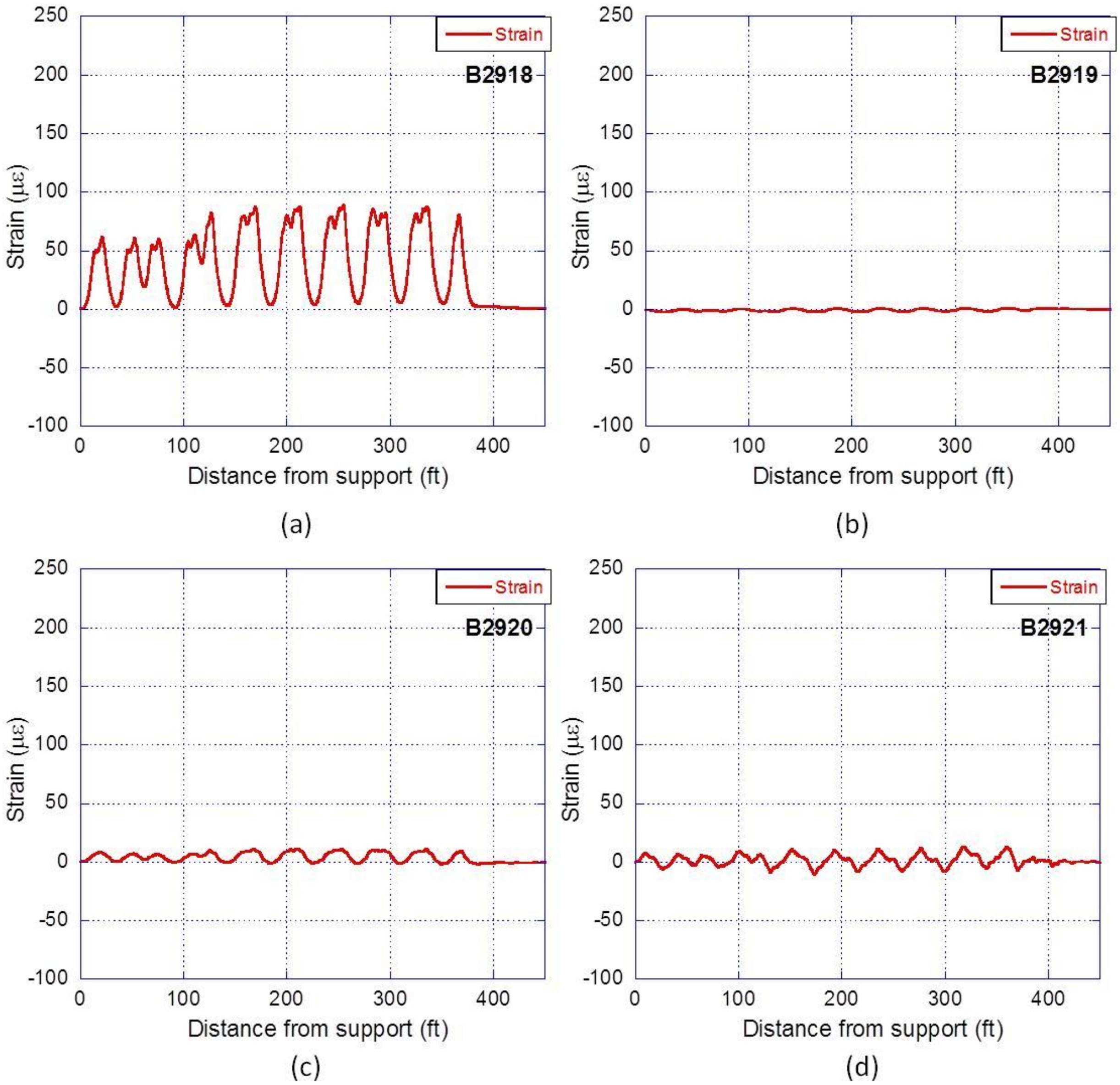


Figure 119: Strain data during Test Run #13 at strain transducer station *(a)* B2918, *(b)* B2919, *(c)* B2920, and *(d)* B2921

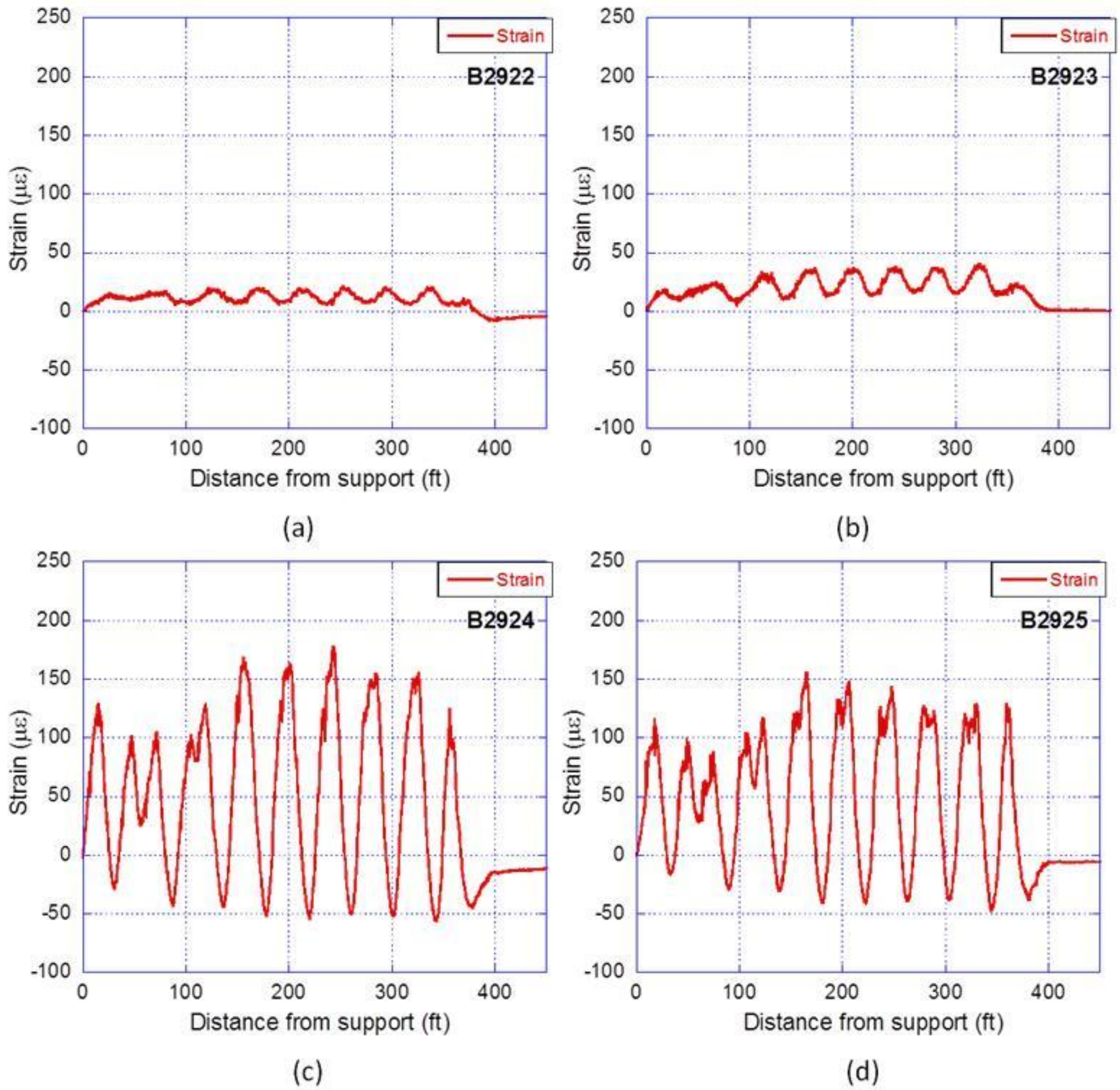


Figure 120: Strain data during Test Run #13 at strain transducer station *(a)* B2922, *(b)* B2923, *(c)* B2924, and *(d)* B2925

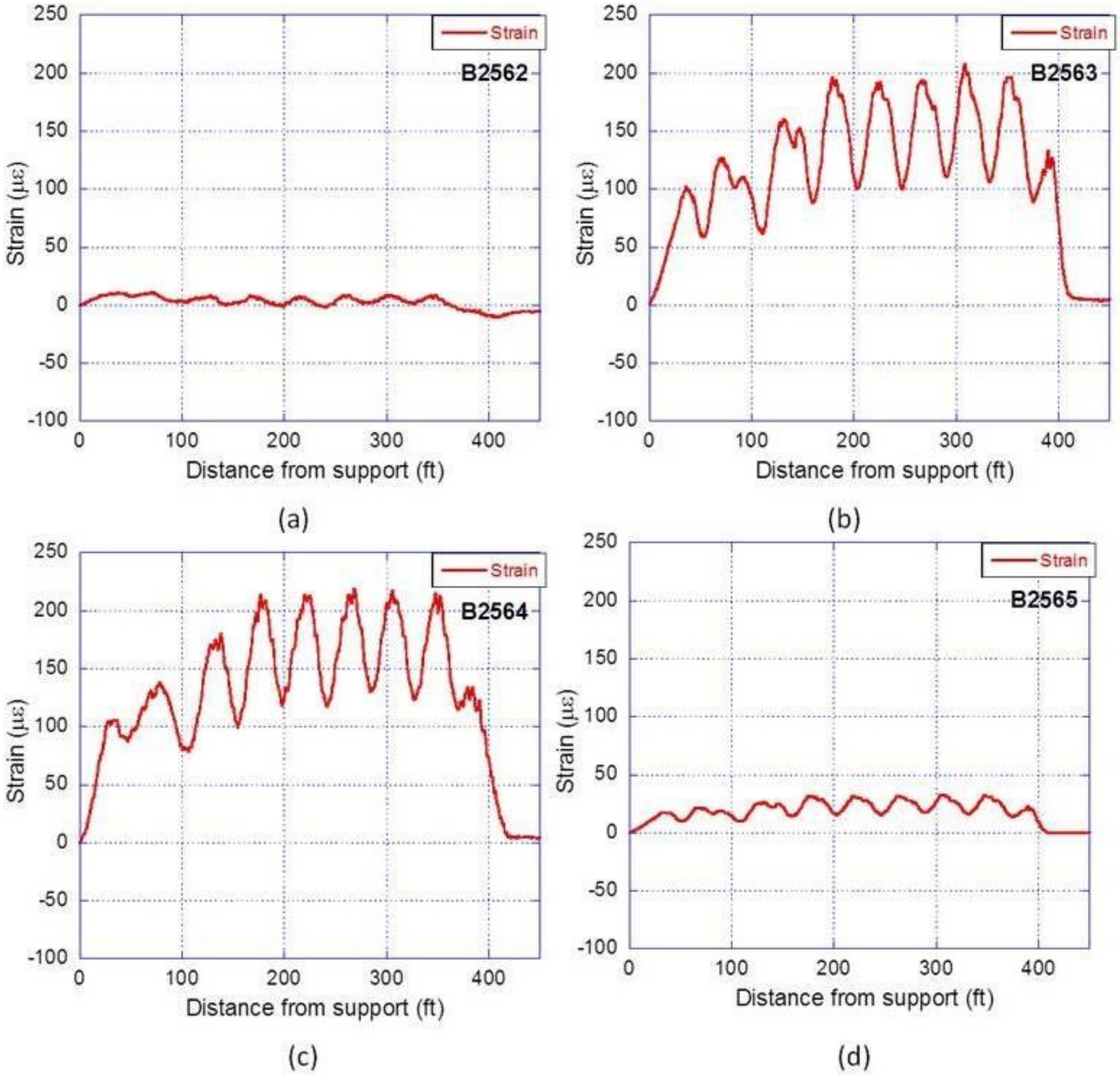


Figure 121: Strain data during Test Run #13 at strain transducer station *(a)* B2562, *(b)* B2563, *(c)* B2564, and *(d)* B2565

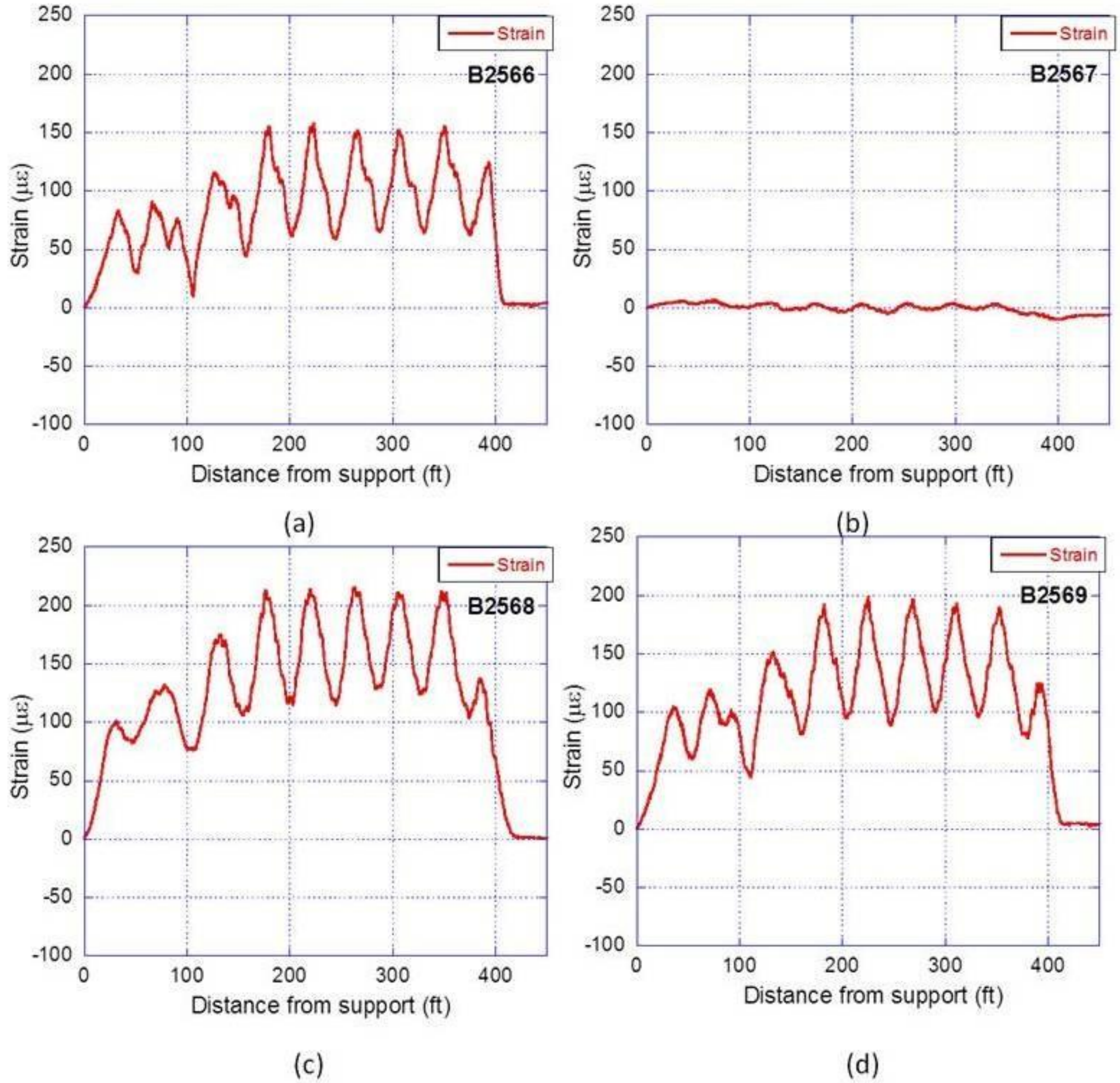


Figure 122: Strain data during Test Run #13 at strain transducer station *(a)* B2566, *(b)* B2567, *(c)* B2568, and *(d)* B2569.

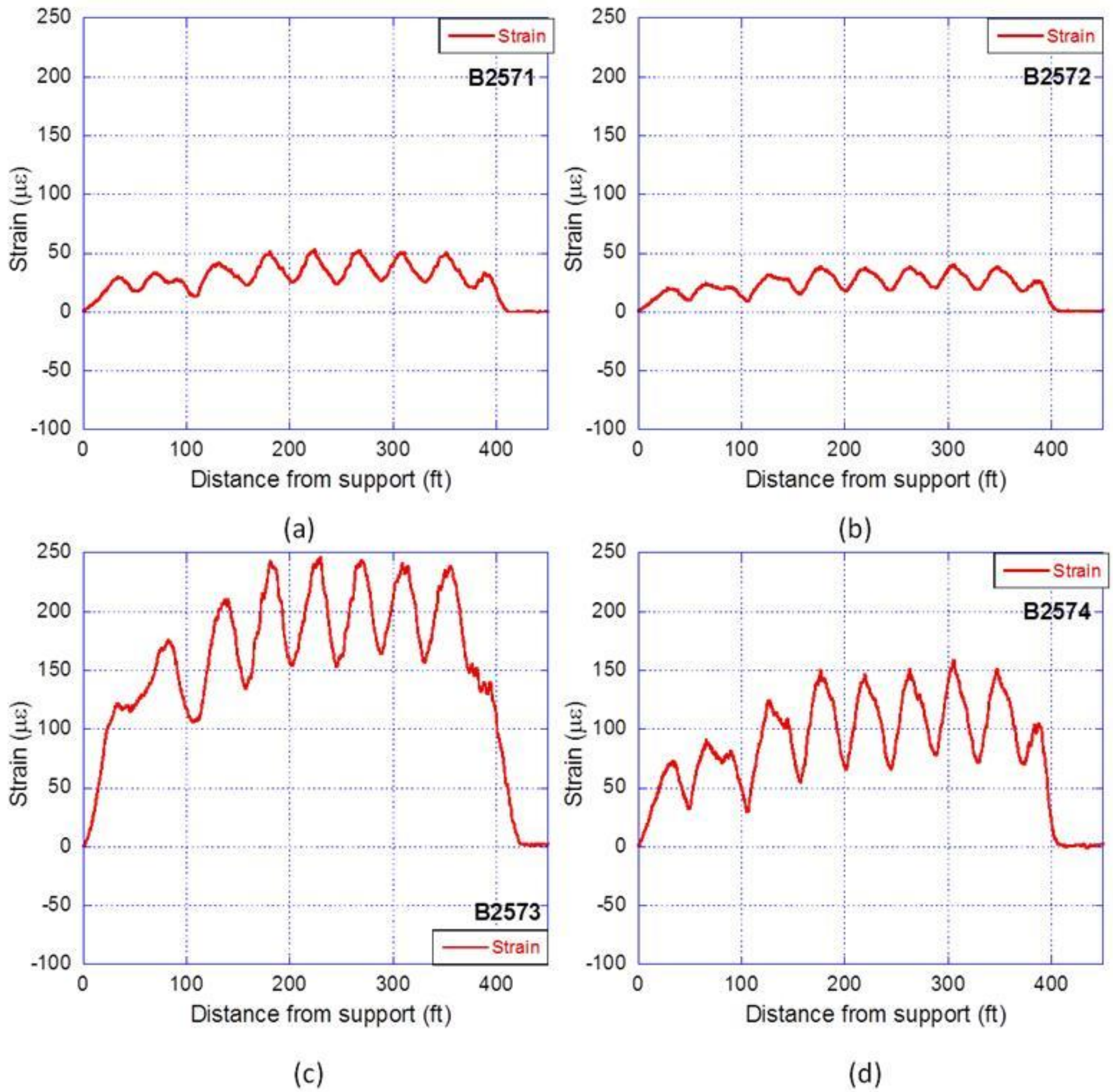


Figure 123: Strain data during Test Run #13 at strain transducer station *(a)* B2571, *(b)* B2572, *(c)* B2573, and *(d)* B2574

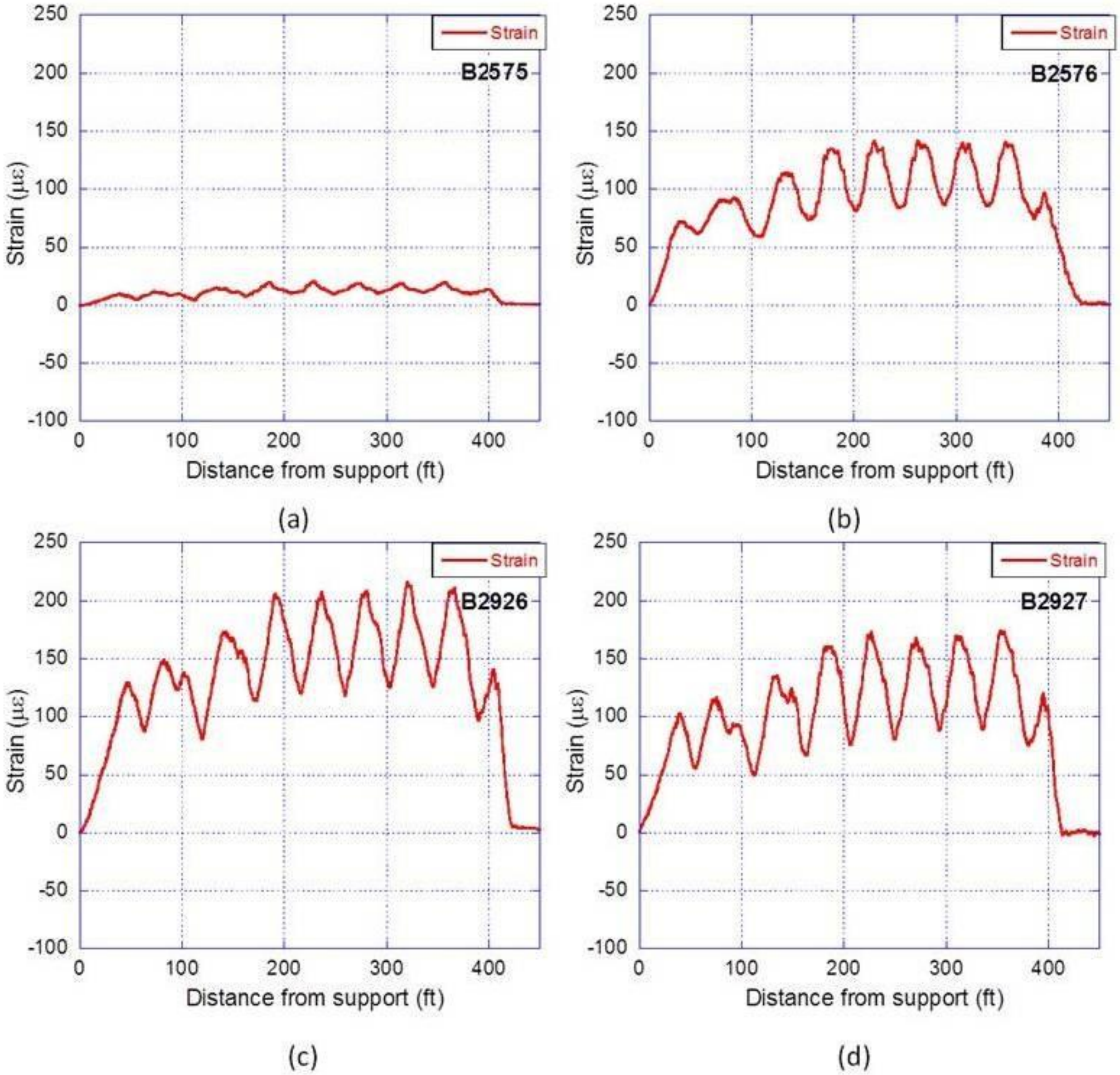


Figure 124: Strain data during Test Run #13 at strain transducer station *(a)* B2575, *(b)* B2576, *(c)* B2926, and *(d)* B2927

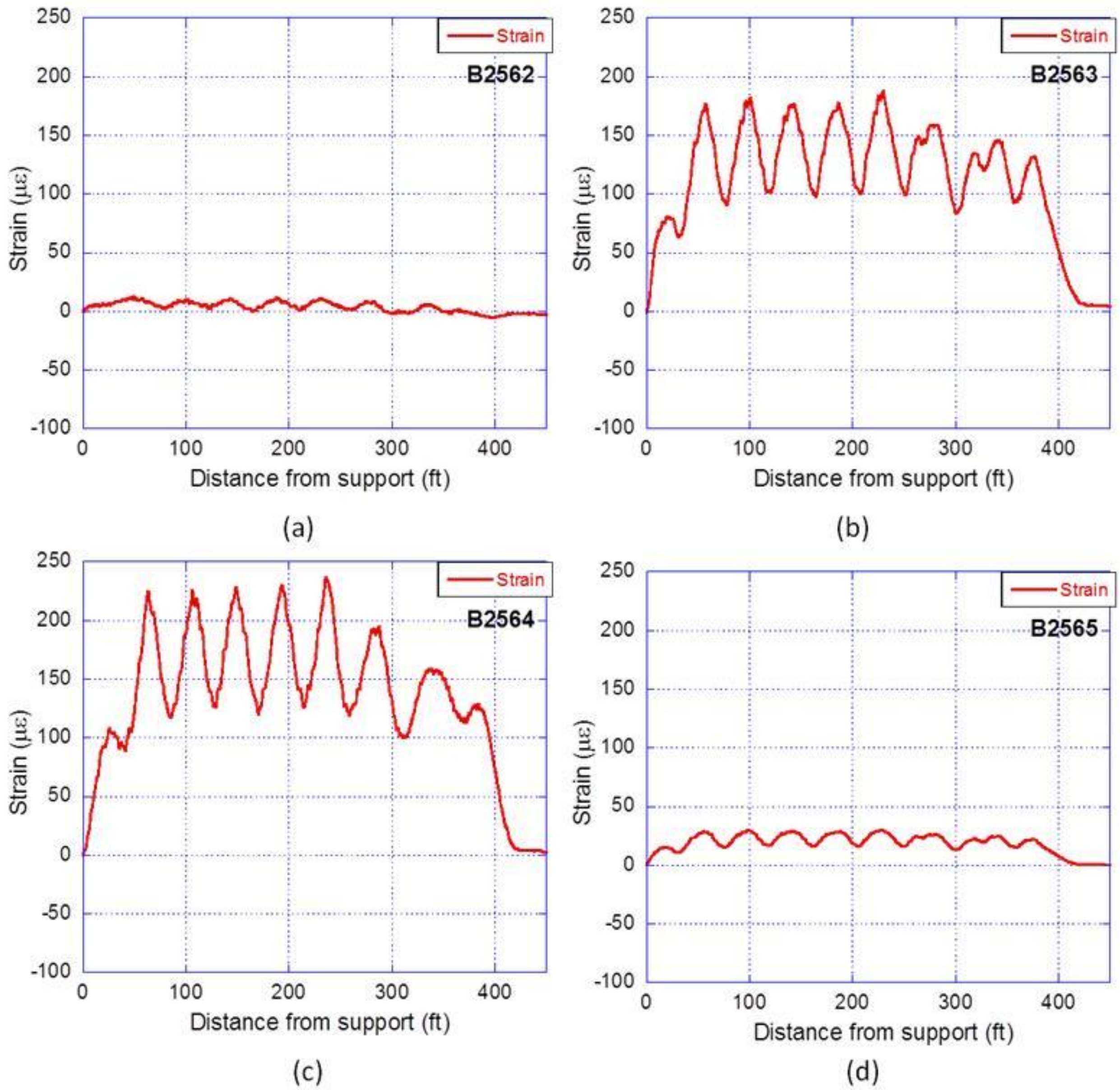


Figure 125: Strain data during Test Run #14 at strain transducer station *(a)* B2562, *(b)* B2563, *(c)* B2564, and *(d)* B2565

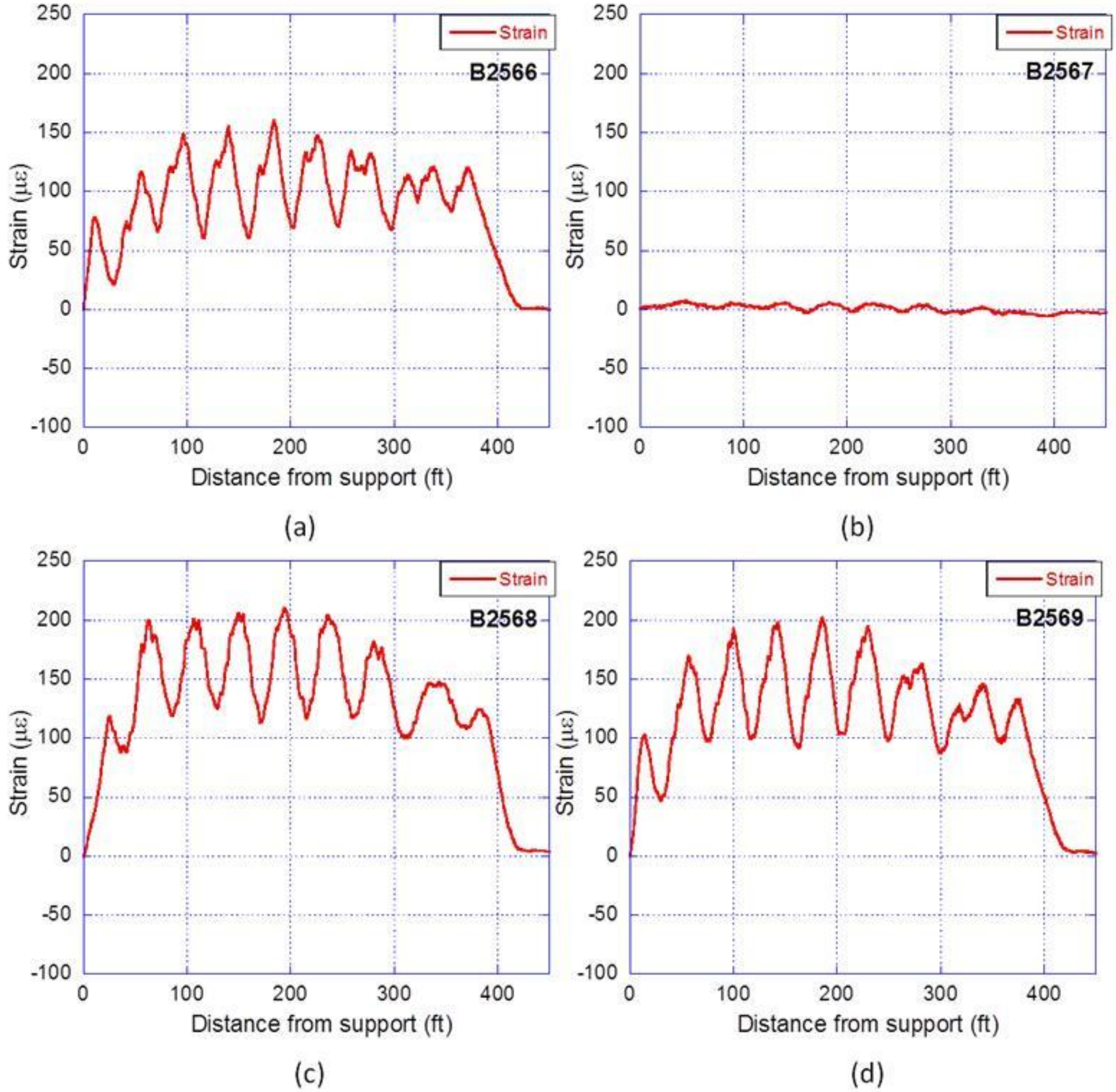


Figure 126: Strain data during Test Run #14 at strain transducer station *(a)* B2566, *(b)* B2567, *(c)* B2568, and *(d)* B2569

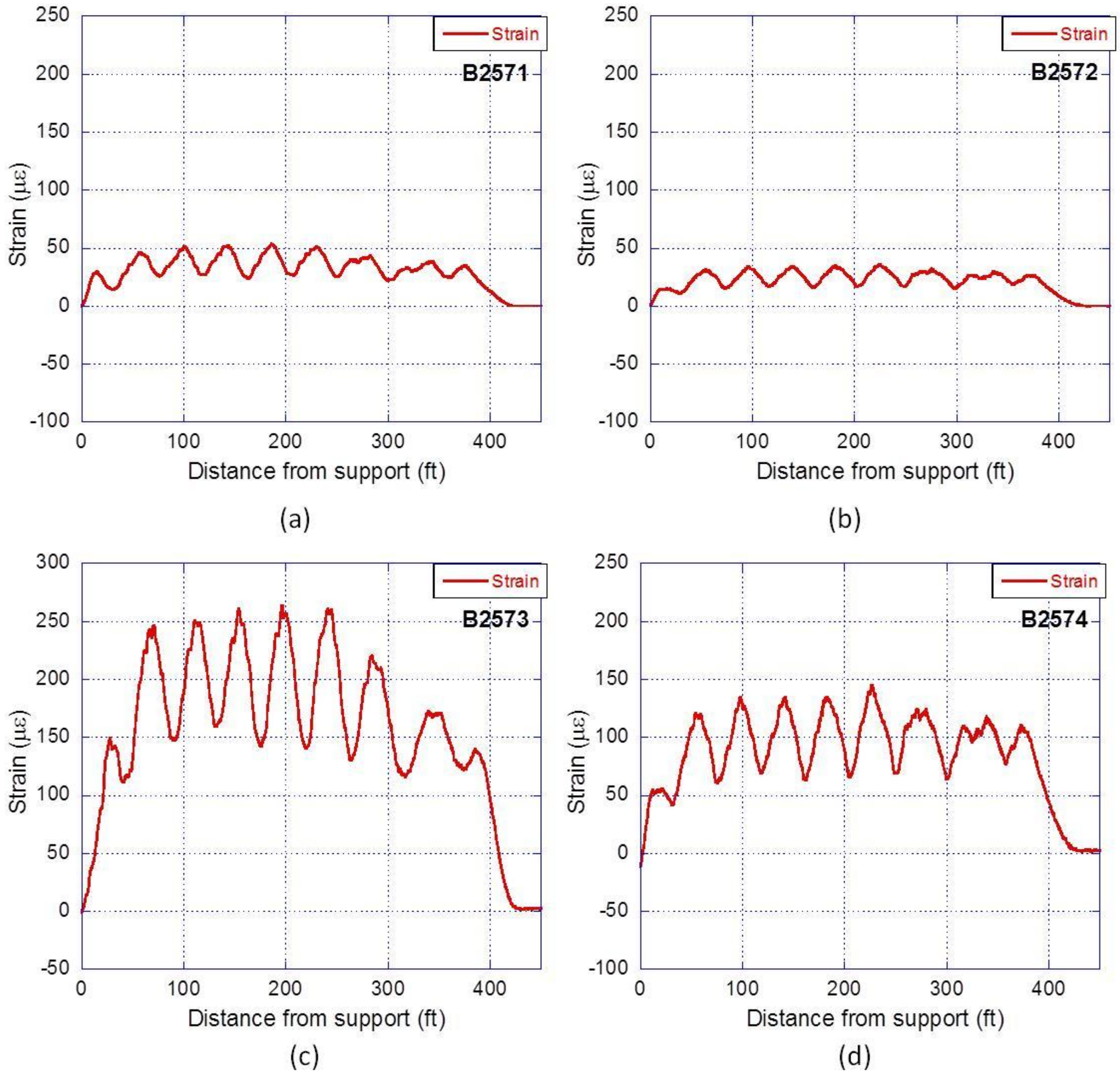


Figure 127: Strain data during Test Run #14 at strain transducer station *(a)* B2571, *(b)* B2572, *(c)* B2573, and *(d)* B2574

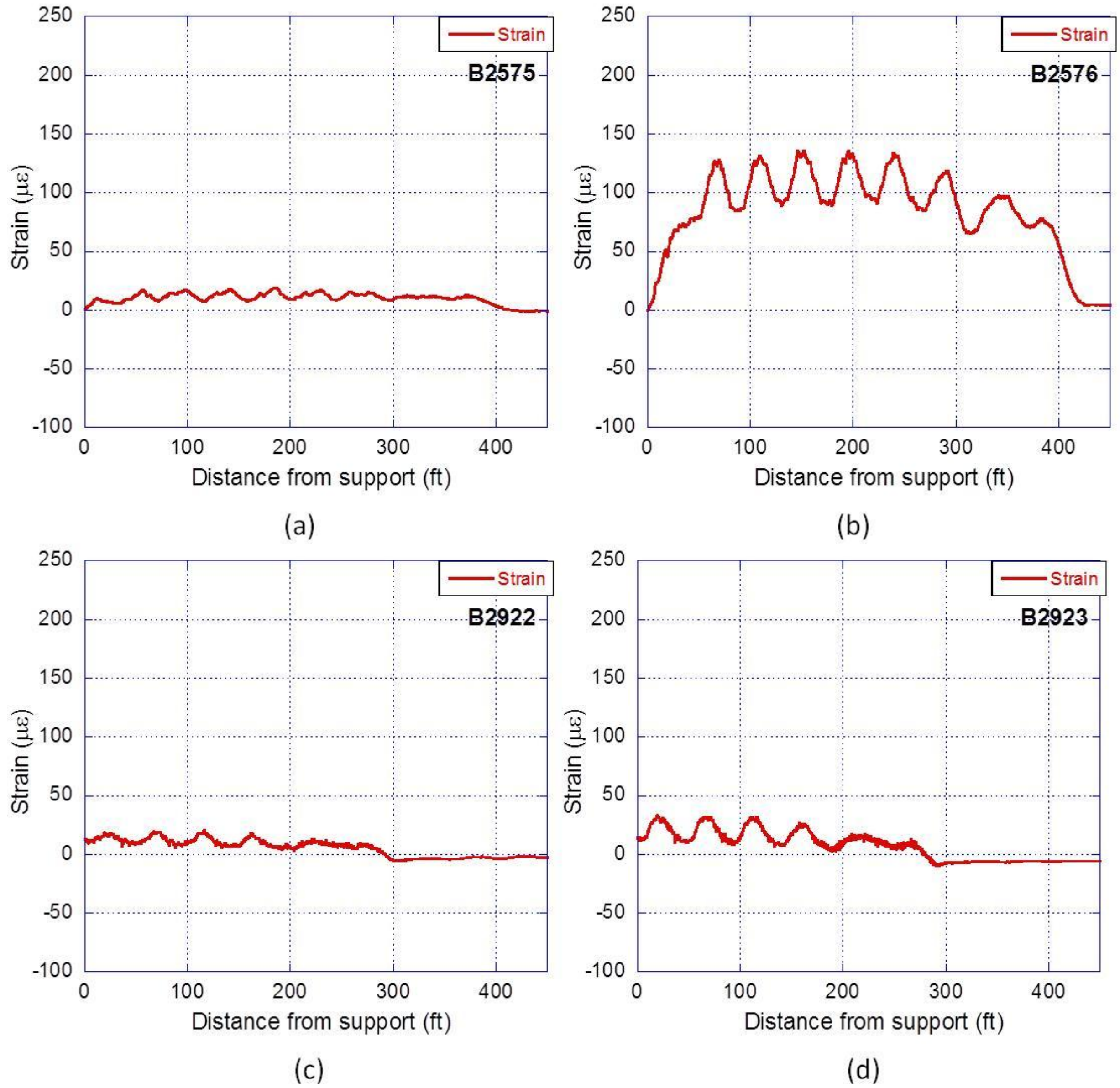


Figure 128: Strain data during Test Run #14 at strain transducer station *(a)* B2575, *(b)* B2576, *(c)* B2922, and *(d)* B2923

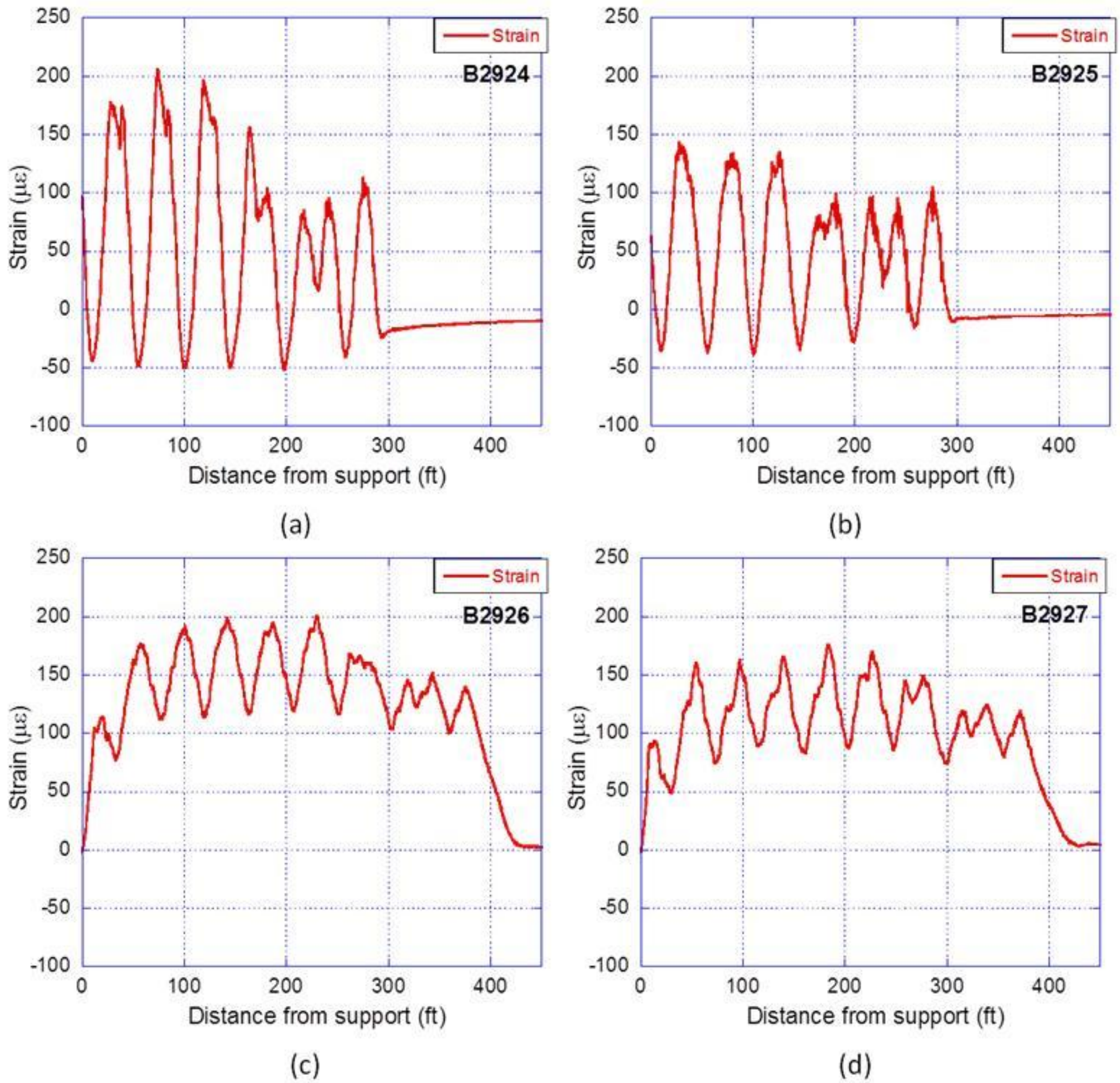


Figure 129: Strain data during Test Run #14 at strain transducer station *(a)* B2924, *(b)* B2925, *(c)* B2926, and *(d)* B2927

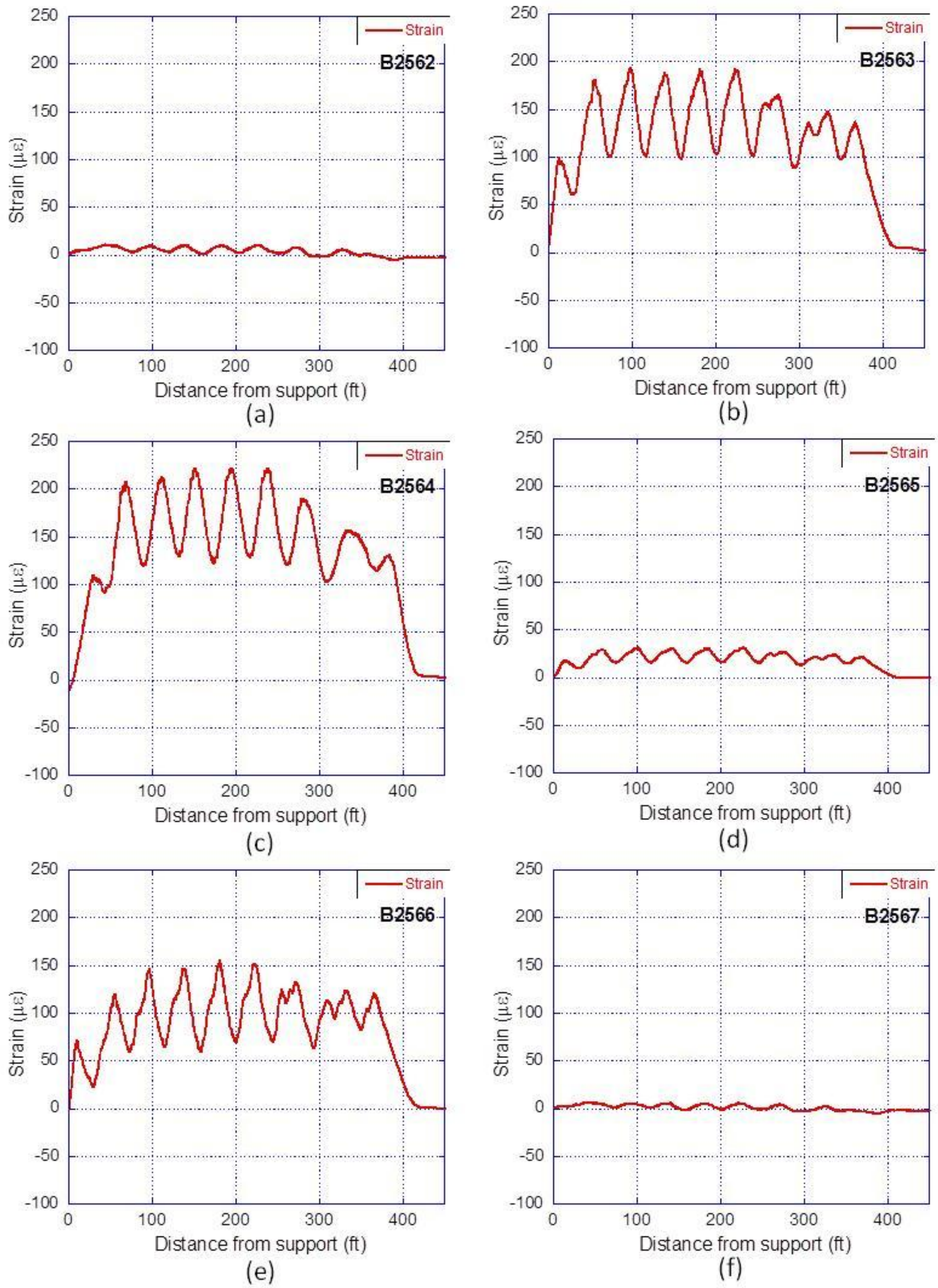


Figure 130.Strain data from Run 16: *(a)* B2562,*(b)* B2563,*(c)* B2564,*(d)* B2565,*(e)* B2566, and*(f)* B2567

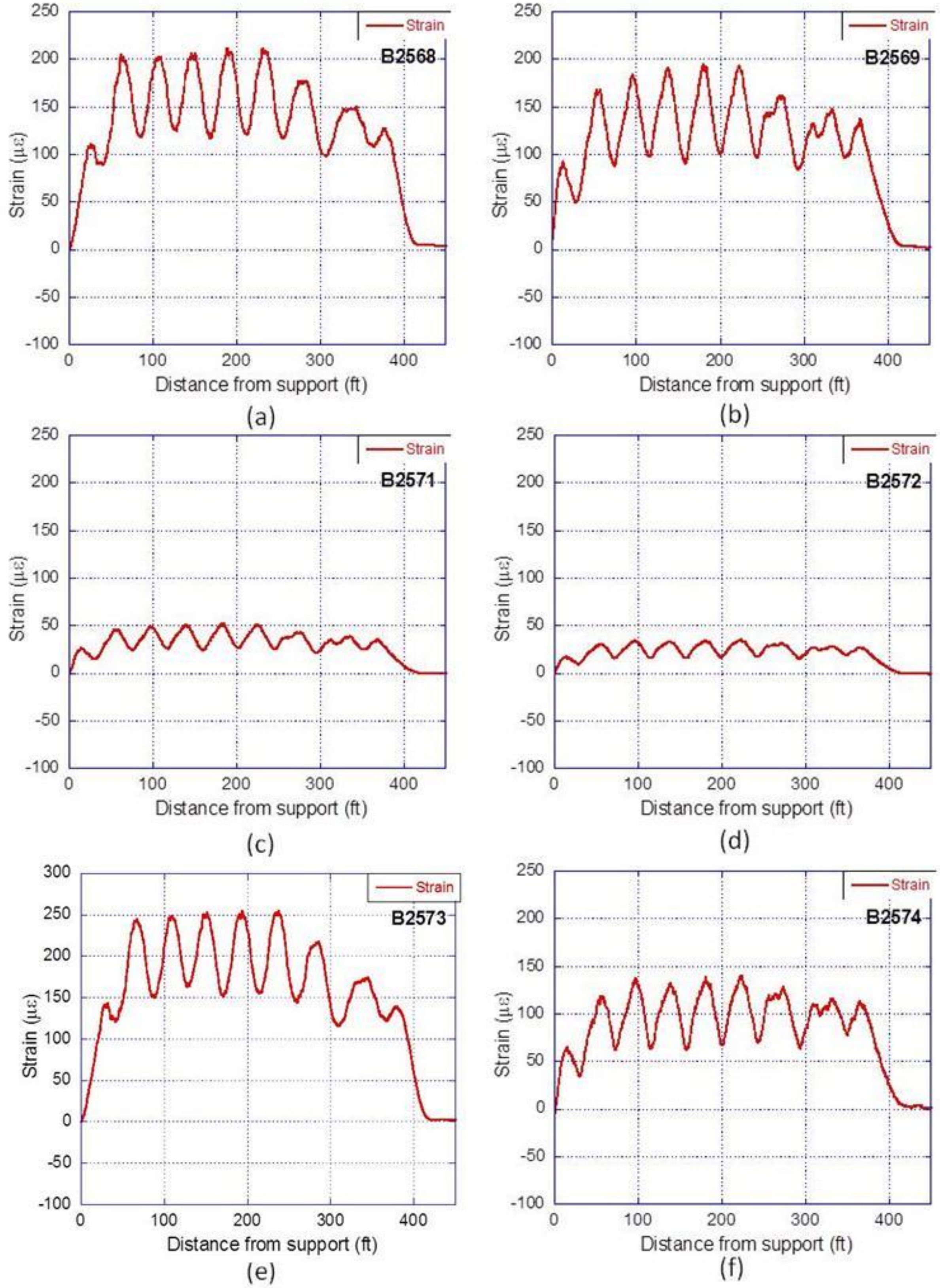


Figure 131.Strain data from Run 16: *(a)* B2568,*(b)* B2569,*(c)* B2571,*(d)* B2572,*(e)* B2573, and *(f)* B2574

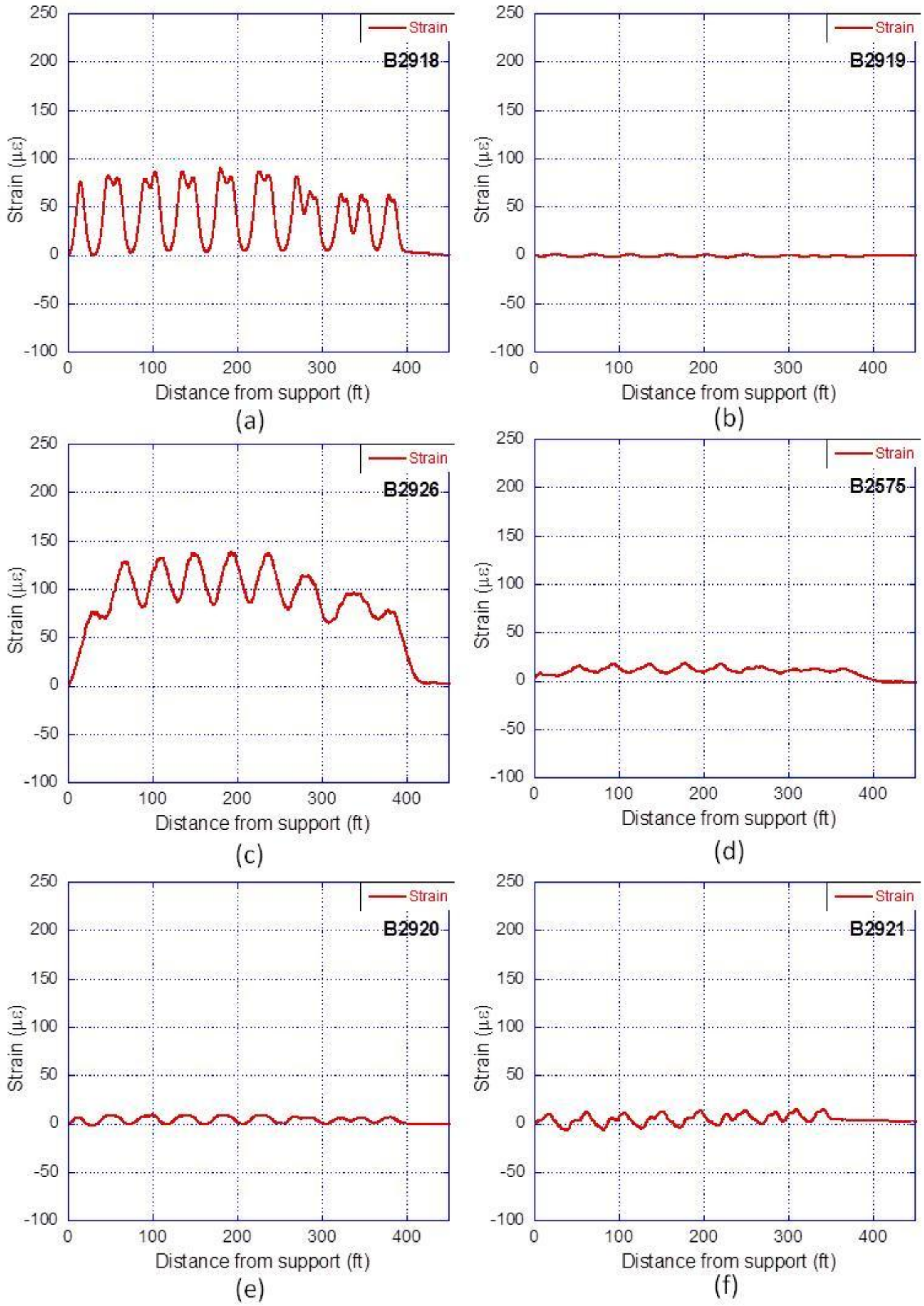


Figure 132.Strain data from Run 16: *(a)* B2918,*(b)* B2919,*(c)* B2926,*(d)* B2575,*(e)* B2920, and *(f)* B2921

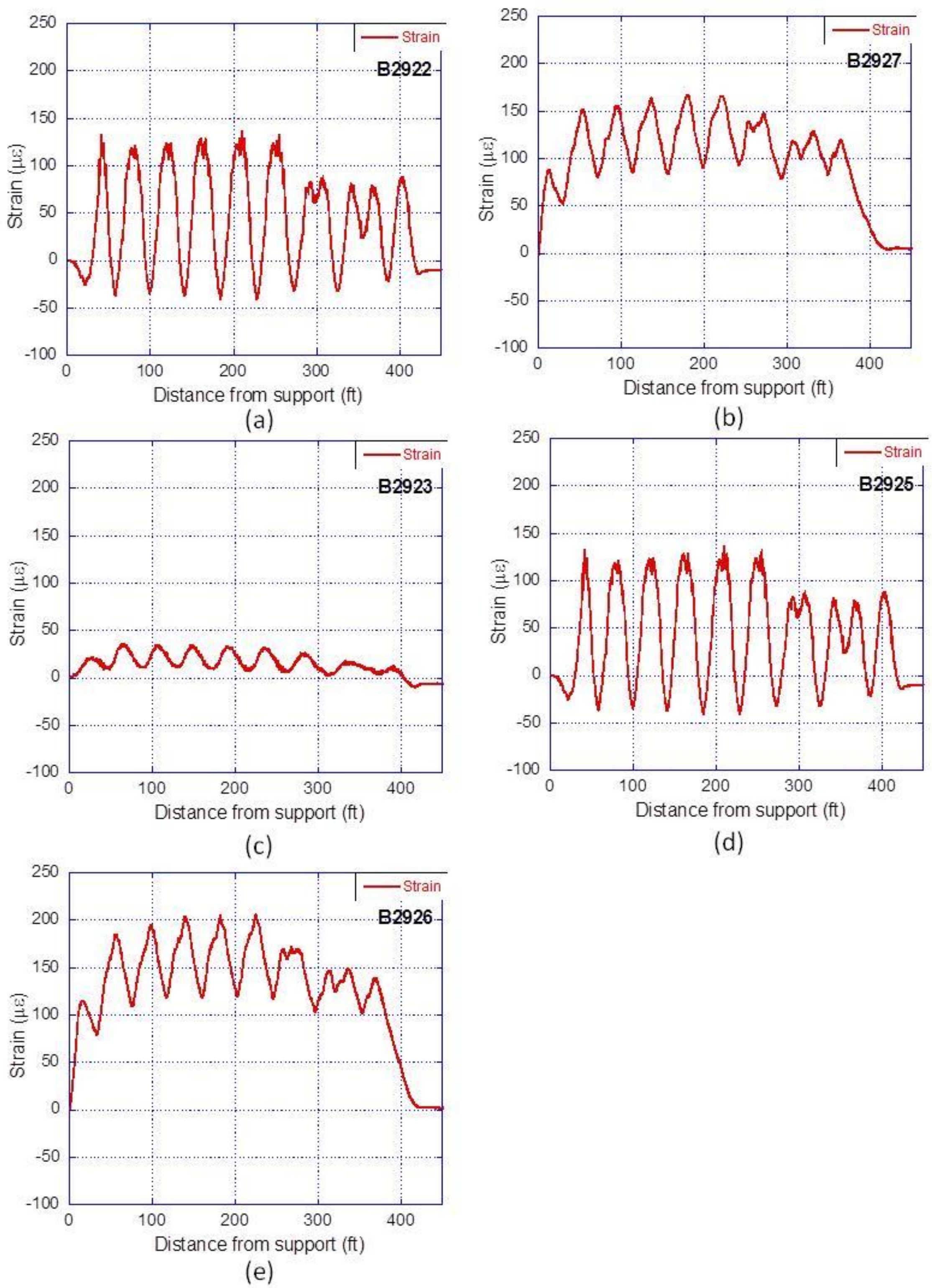


Figure 133.Strain data from Run 16: *(a)* B2922,*(b)* B2927,*(c)* B2923,*(d)* B2925, and *(e)* B2926.

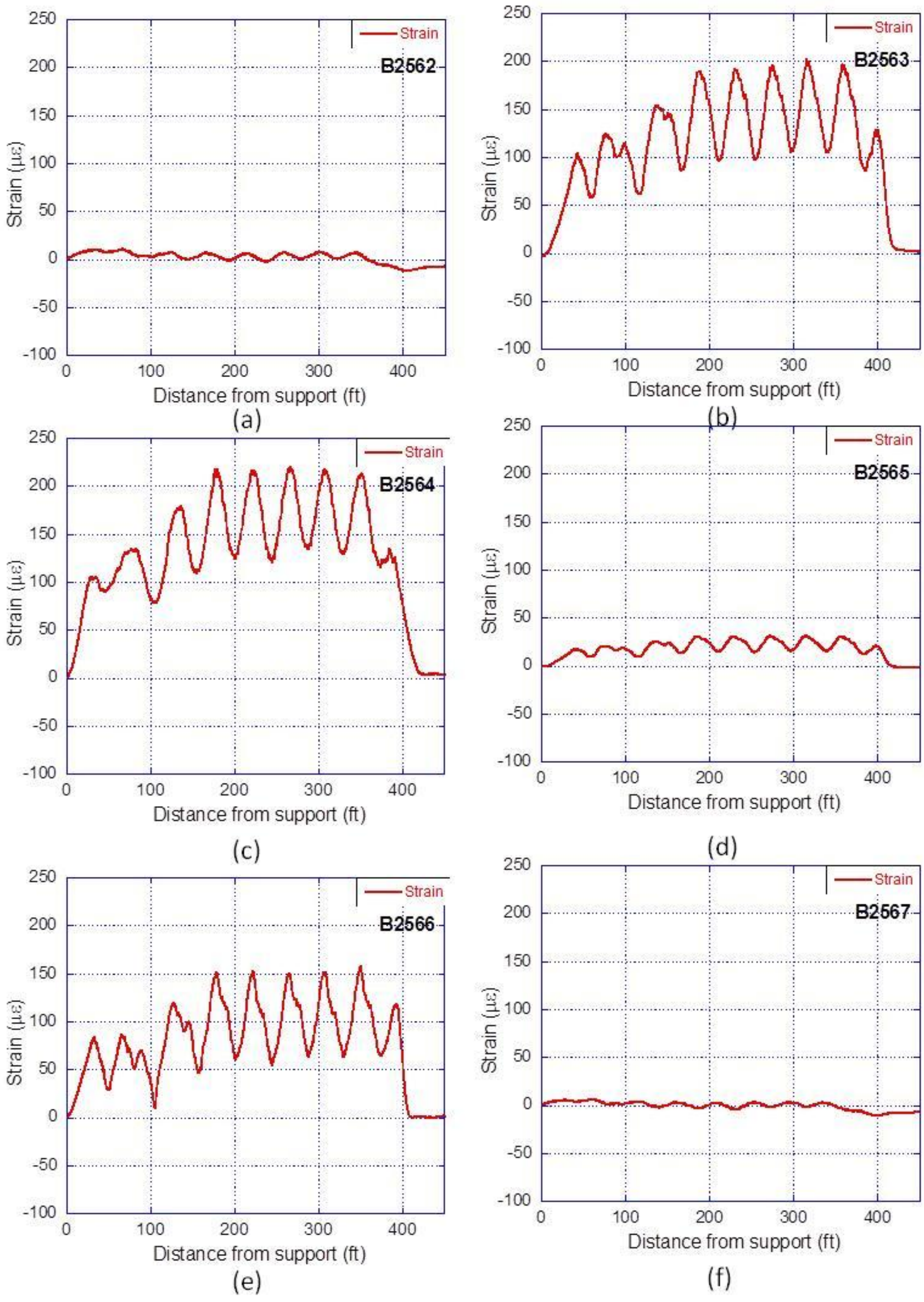


Figure 134.Strain data from Run 17: *(a)* B2562,*(b)* B2563,*(c)* B2564,*(d)* B2565,*(e)* B2566, and *(f)* B2567

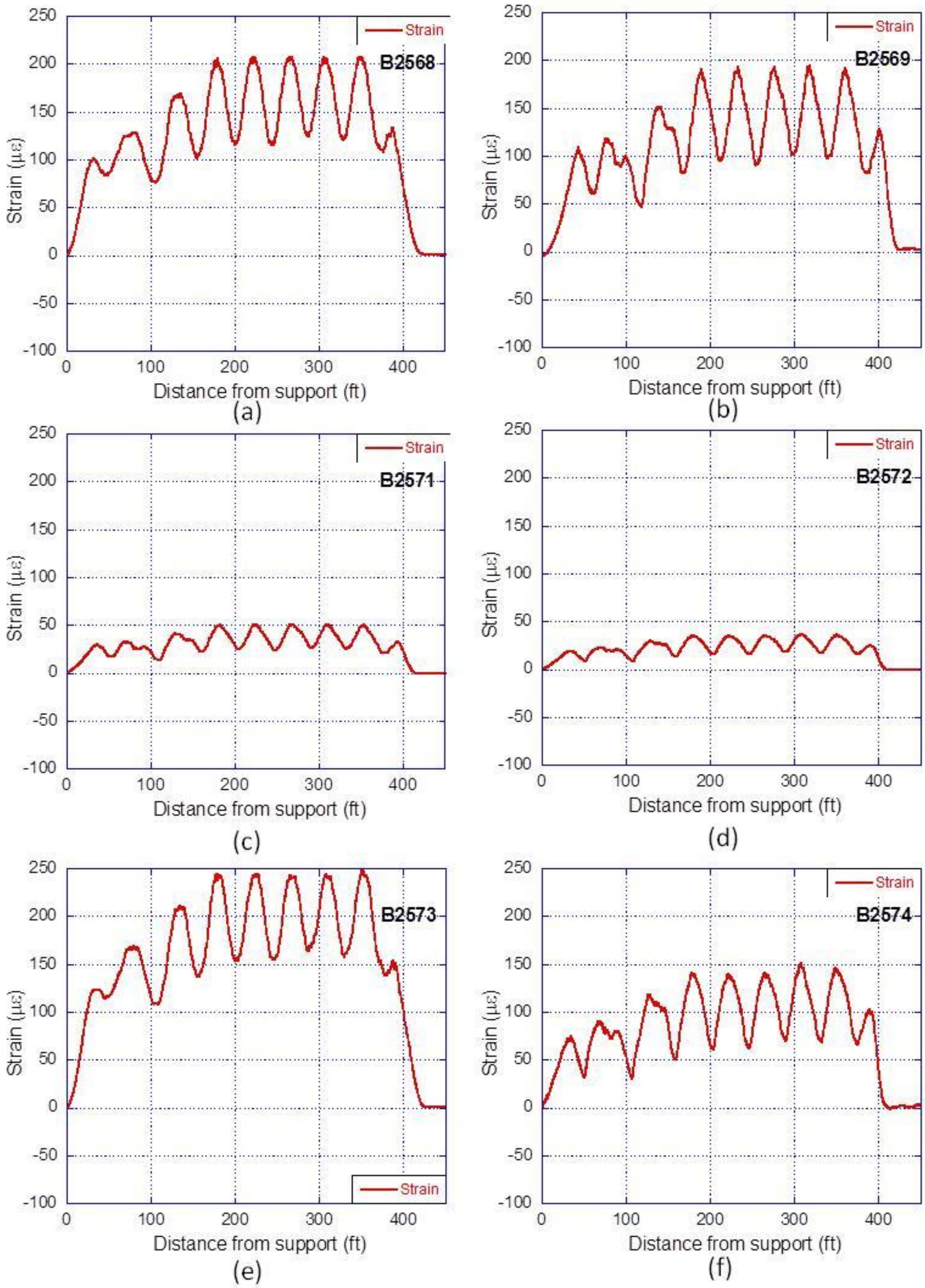


Figure 135.Strain data from Run 17: *(a)* B2568,*(b)* B2569,*(c)* B2571,*(d)* B2572,*(e)* B2573, and *(f)* B2574

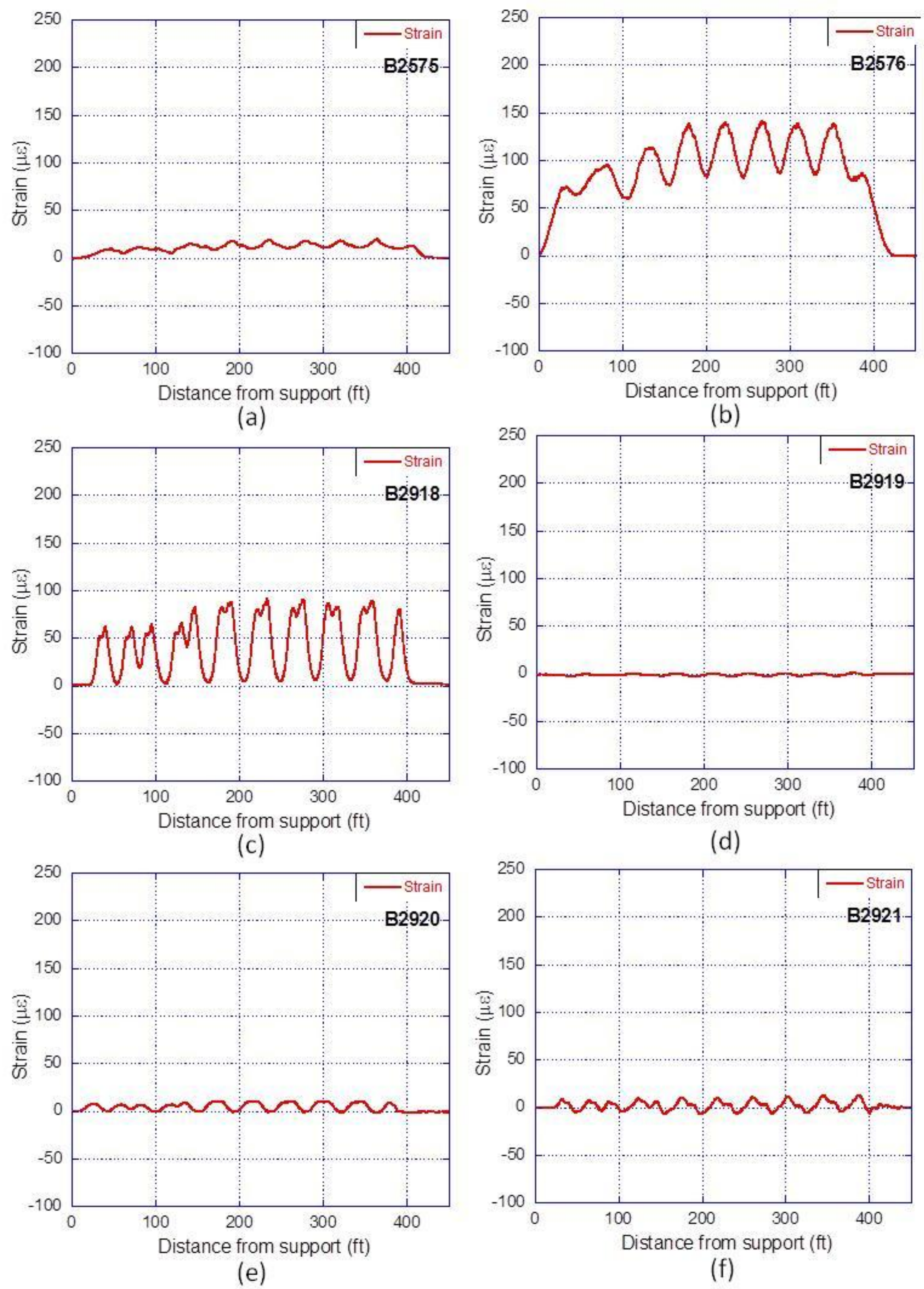


Figure 136.Strain data from Run 17: *(a)* B2575,*(b)* B2576,*(c)* B2918,*(d)* B2919,*(e)* B2920, and *(f)* B2921

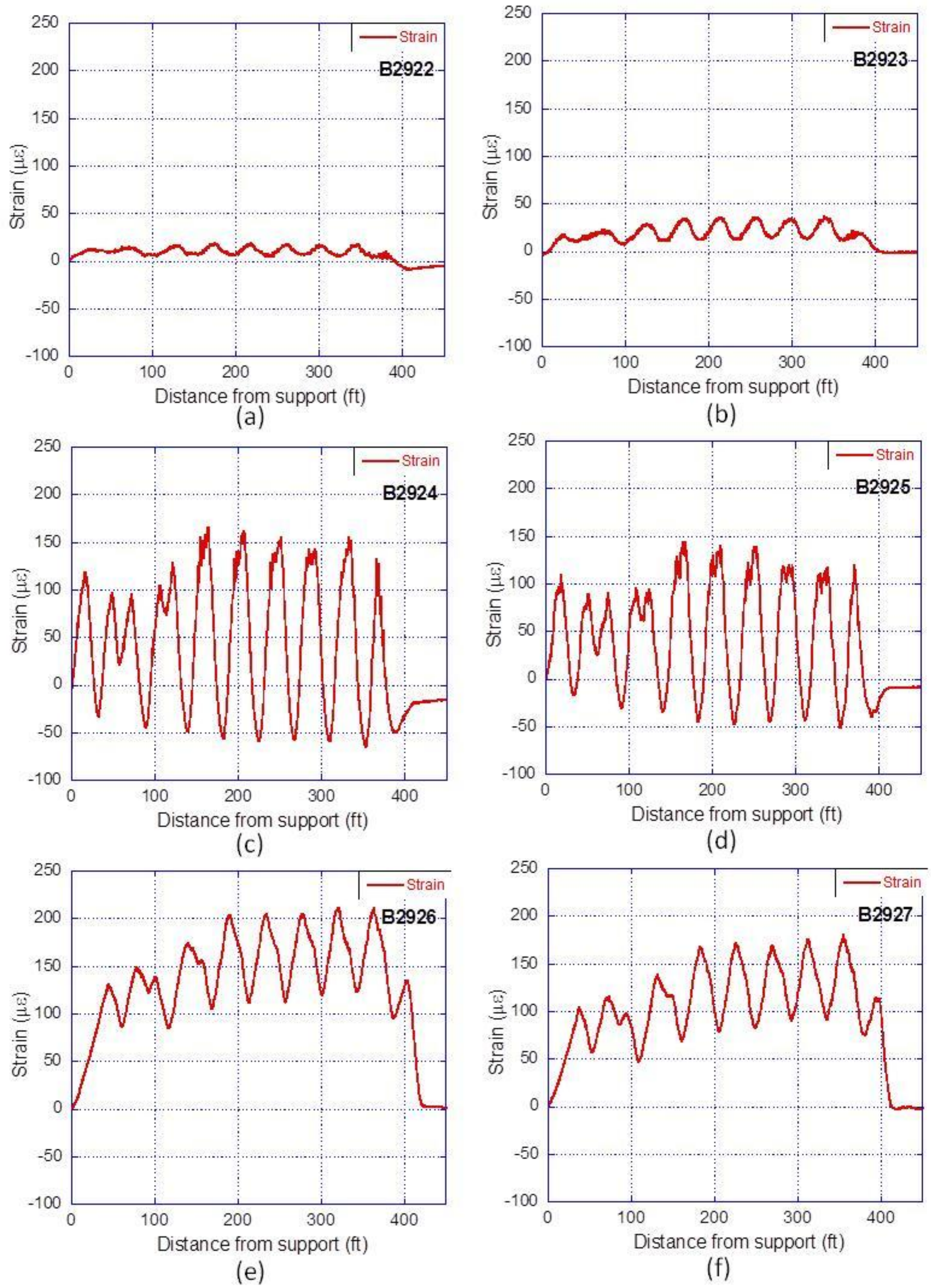


Figure 137.Strain data from Run 17: *(a)* B2922,*(b)* B2923,*(c)* B2924,*(d)* B2925,*(e)* B2926, and*(f)* B2927

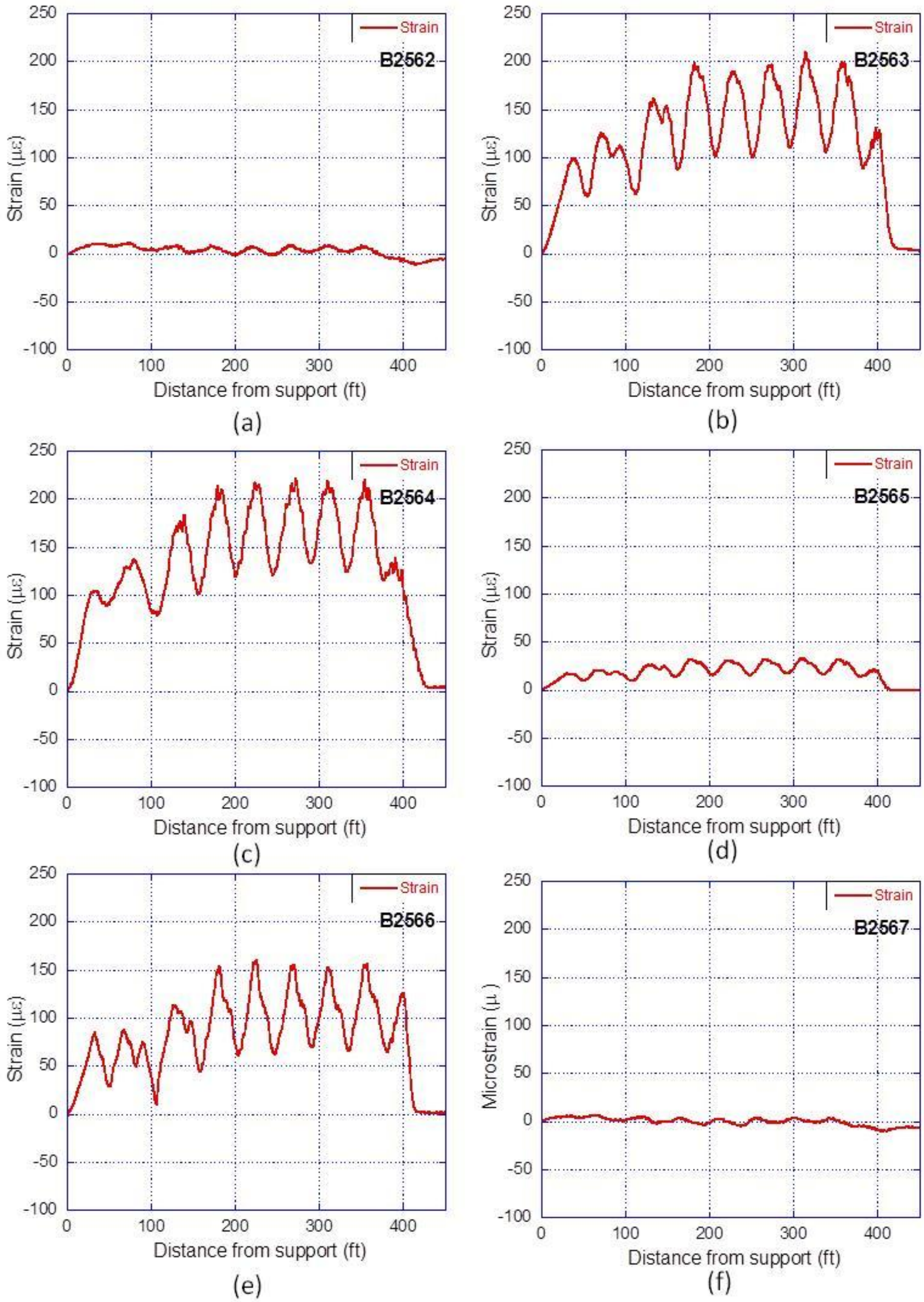


Figure 138.Strain data from Run 19: *(a)* B2562,*(b)* B2563,*(c)* B2564,*(d)* B2565,*(e)* B2566, and*(f)* B2567

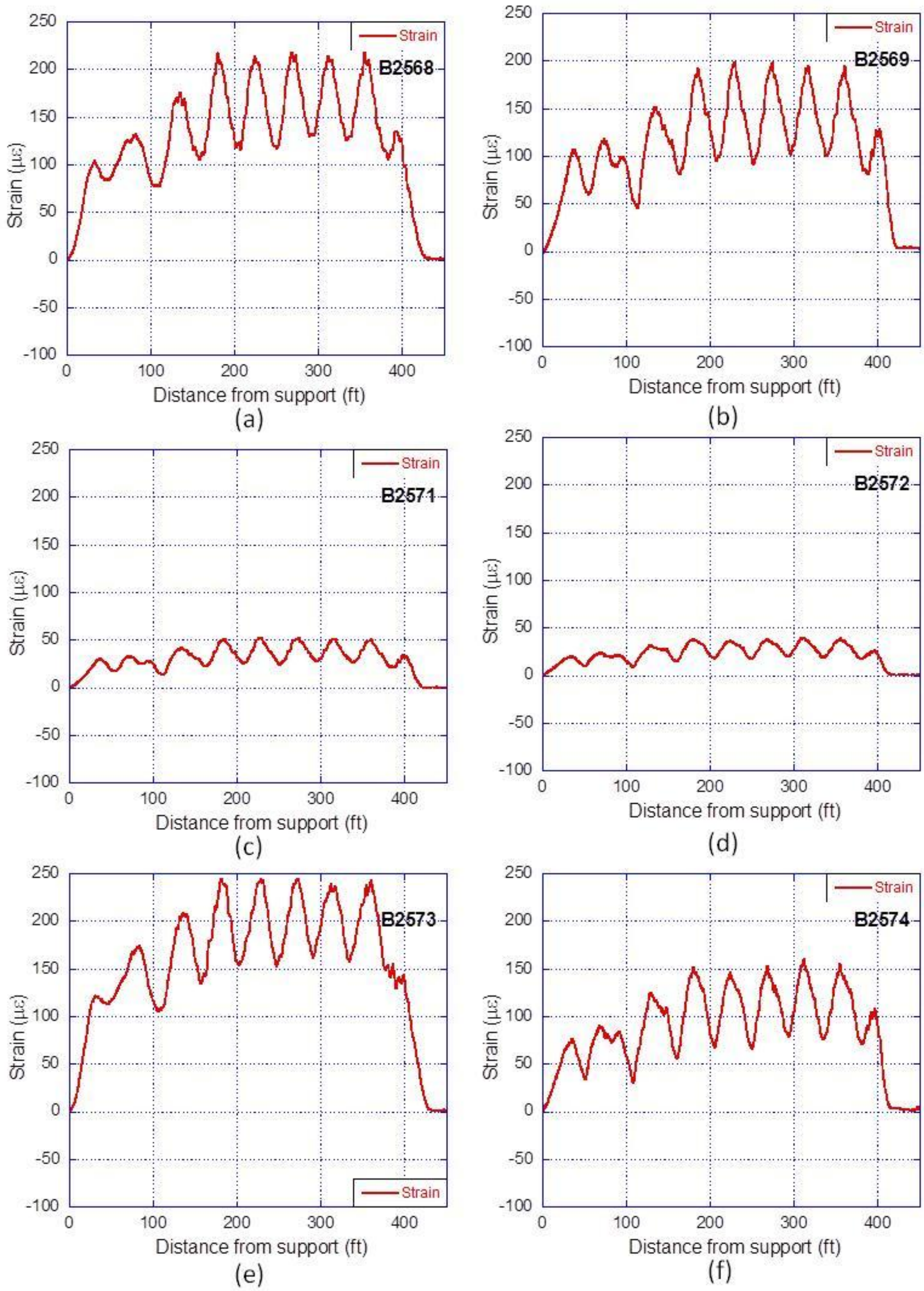


Figure 139.Strain data from Run 19: *(a)* B2568,*(b)* B2569,*(c)* B2571,*(d)* B2572,*(e)* B2573, and*(f)* B2574

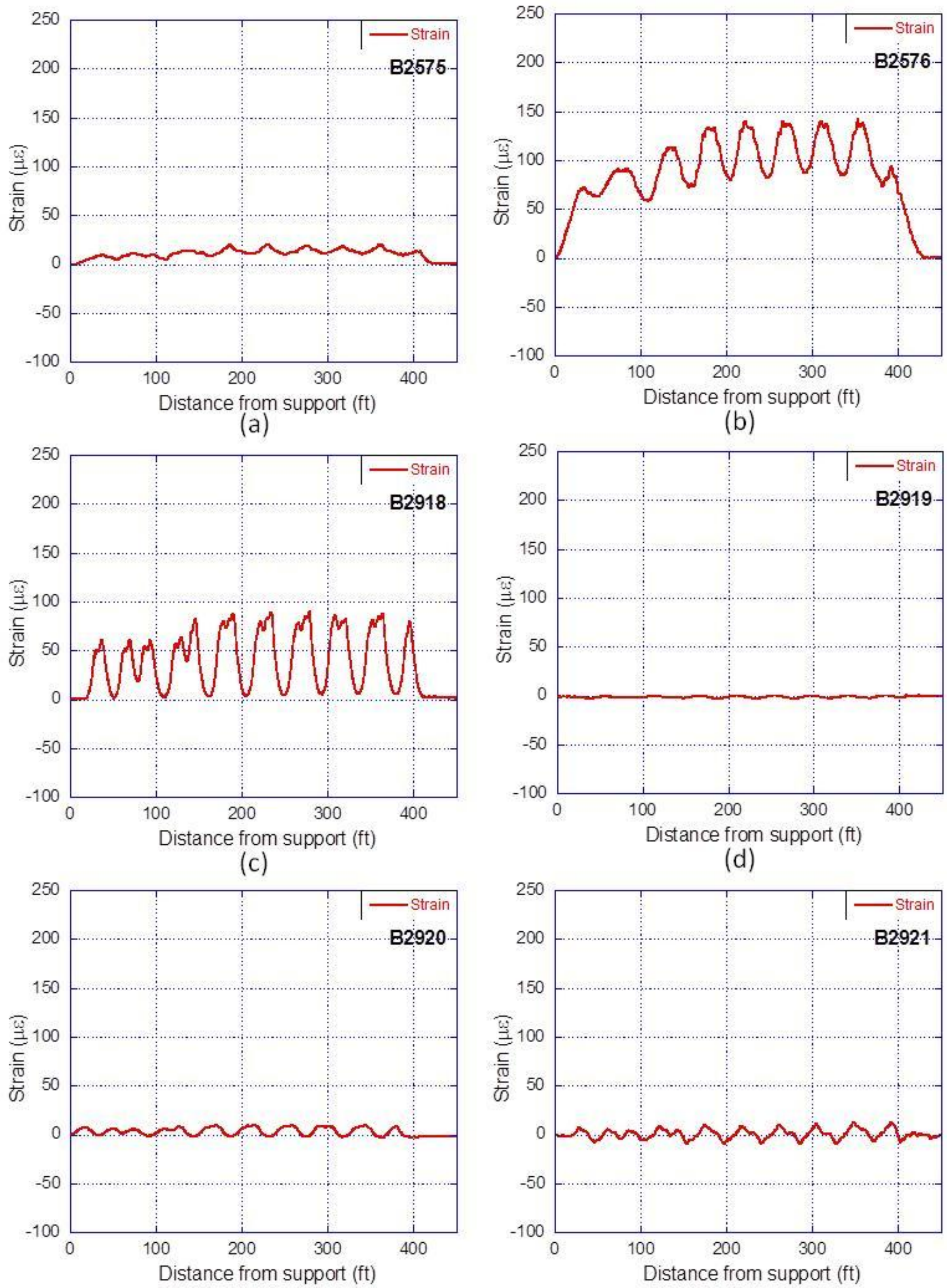


Figure 140.Strain data from Run 19: *(a)* B2575; *(b)* B2576,*(c)* B2918,*(d)* B2919,*(e)* B2920, and*(f)* B2921

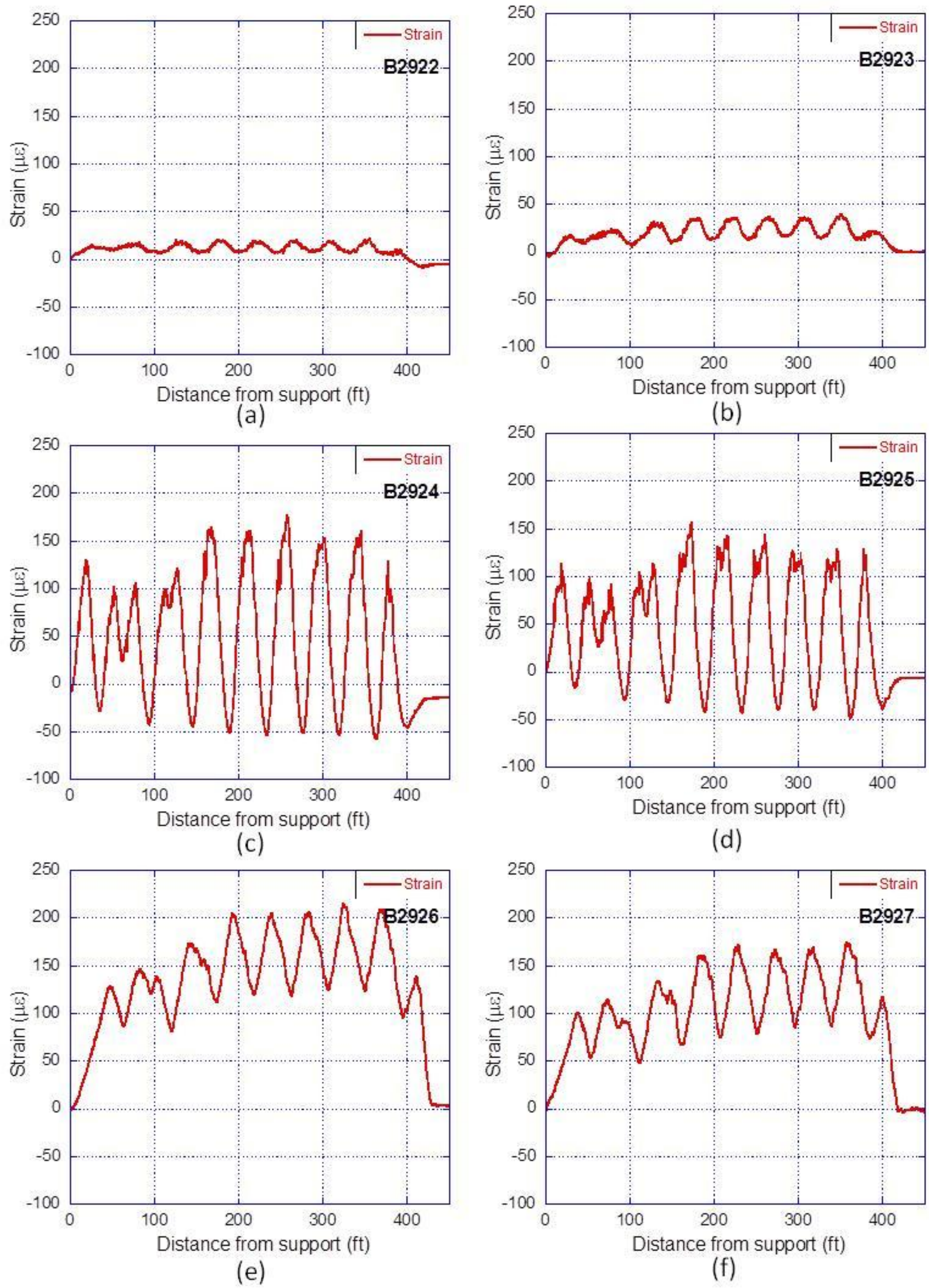


Figure 141.Strain data from Run 19: *(a)* B2922;*(b)* B2923,*(c)* B2924,*(d)* B2925,*(e)* B2926, and*(f)* B2927

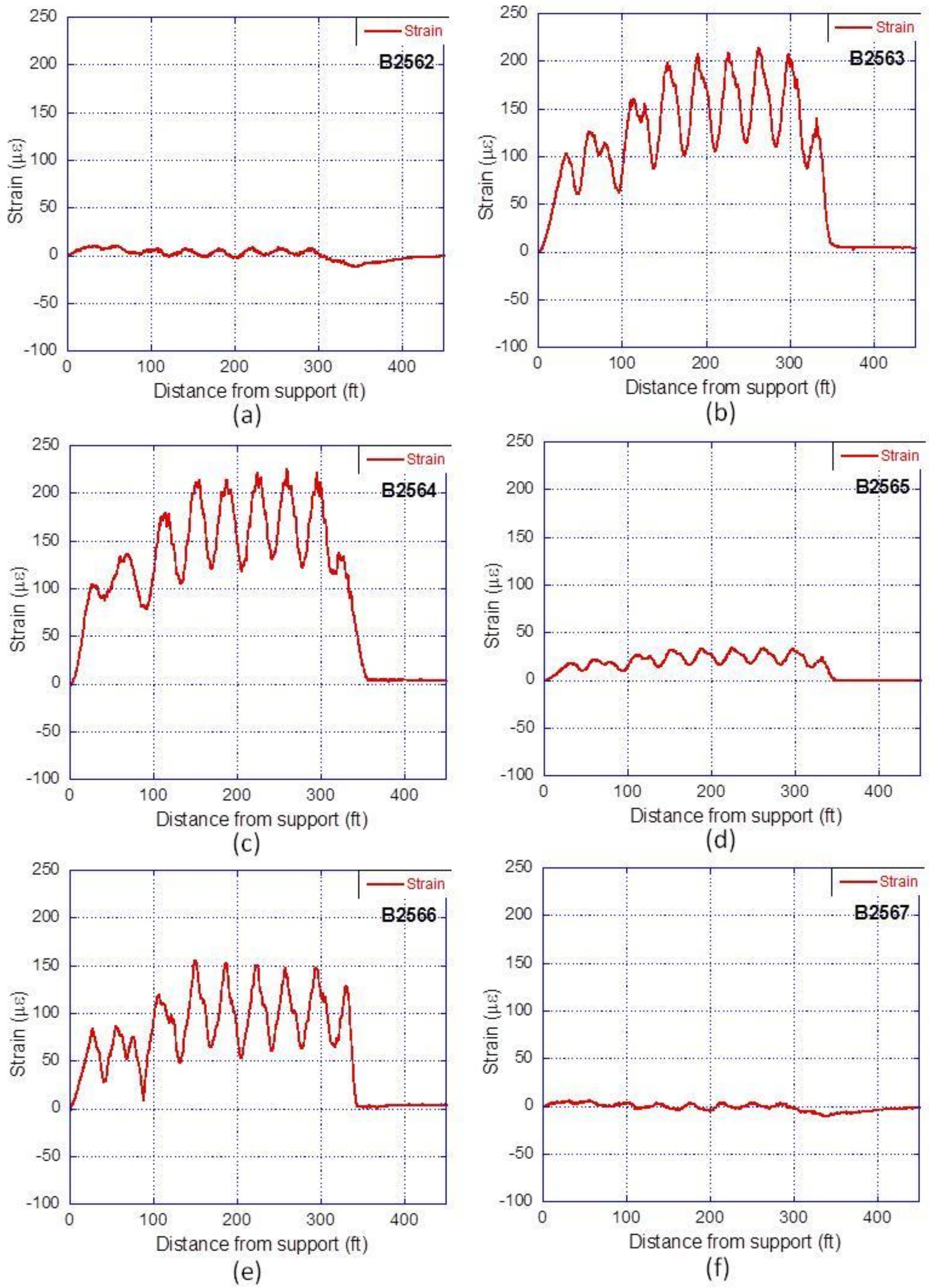


Figure 142.Strain data from Run 21: *(a)* B2562,*(b)* B2563,*(c)* B2564,*(d)* B2565,*(e)* B2566, and*(f)* B2567

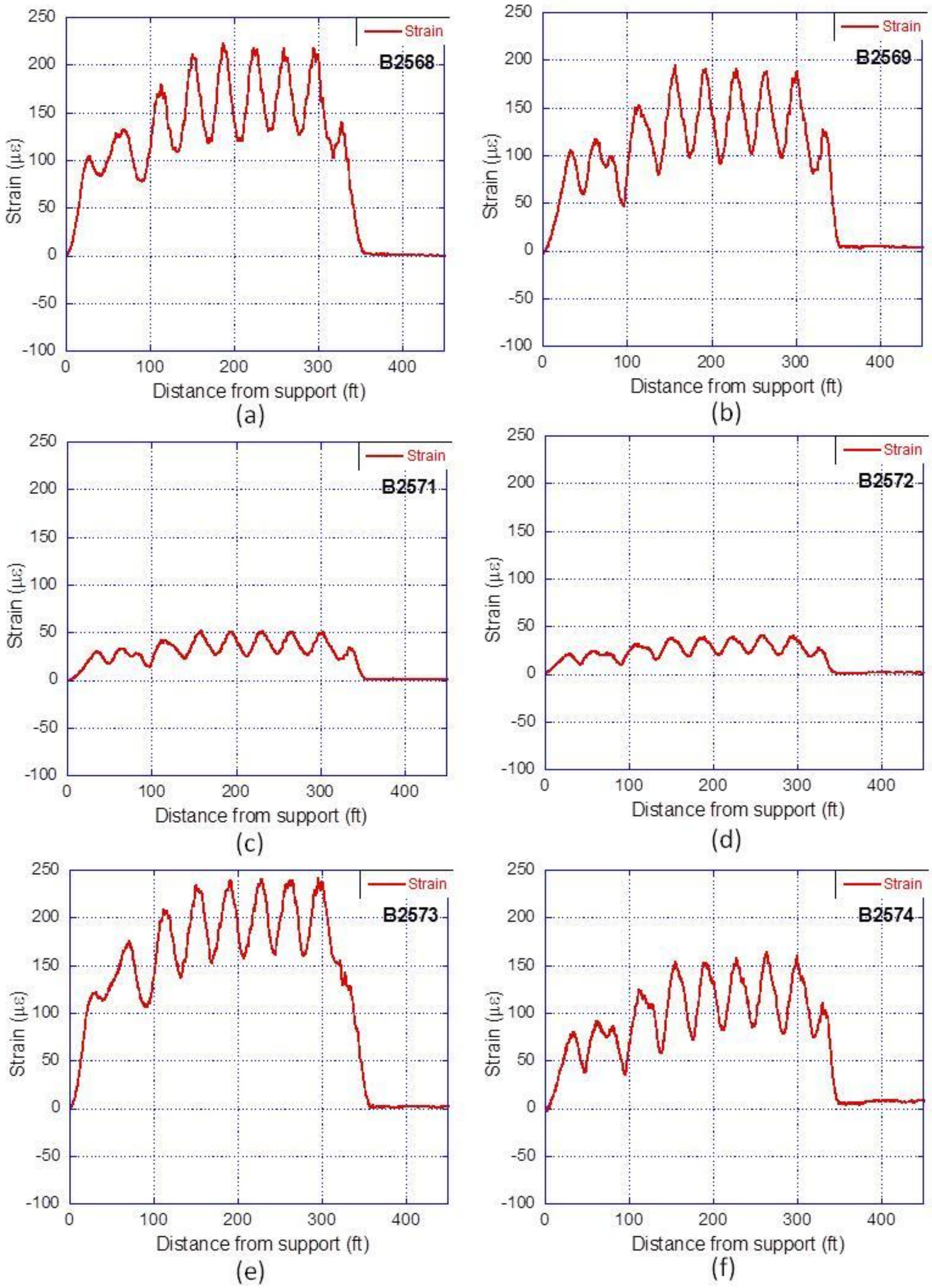


Figure 143.Strain data from Run 21: *(a)* B2568,*(b)* B2569,*(c)* B2571,*(d)* B2572,*(e)* B2573, and*(f)* B2574

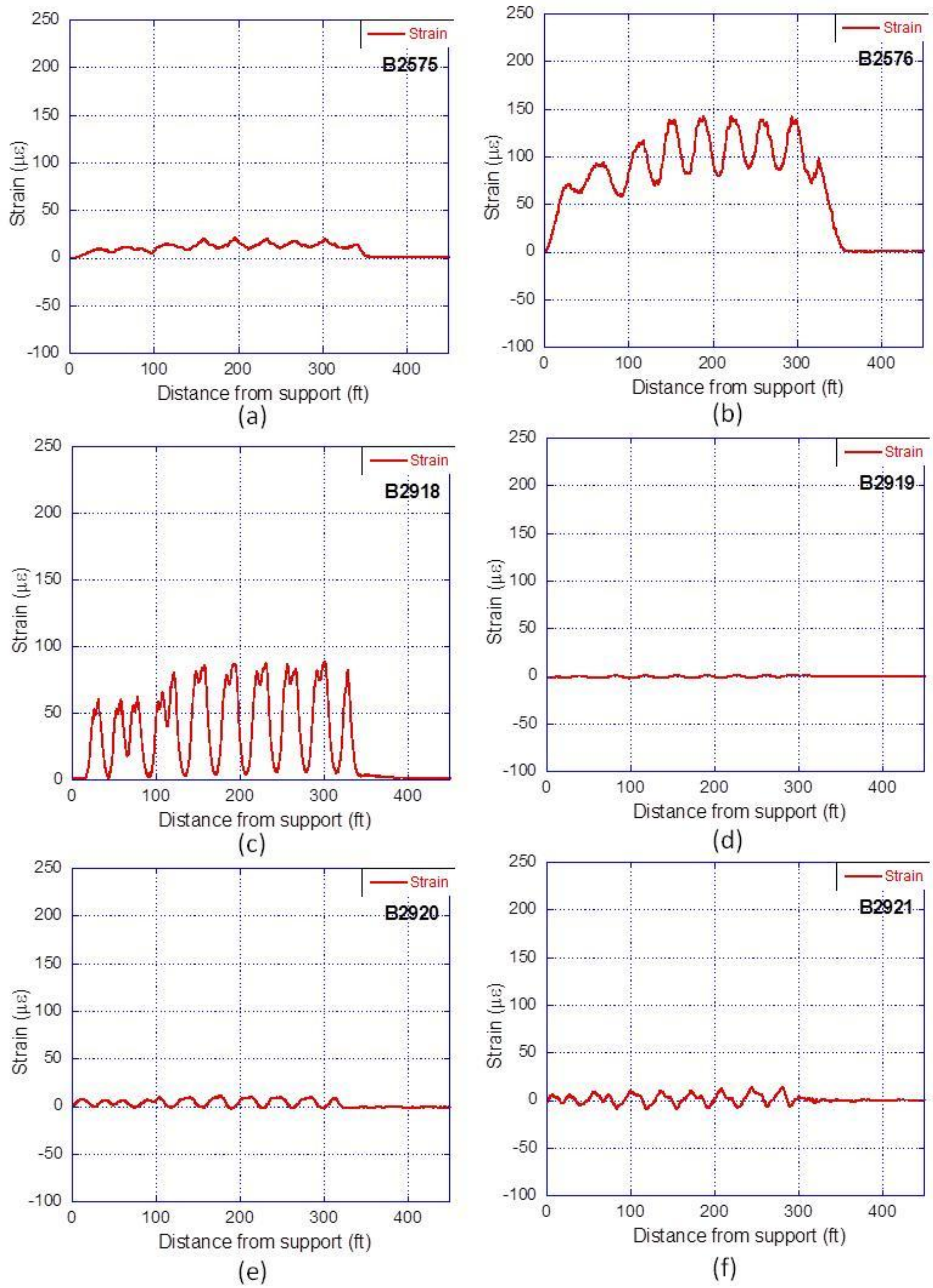


Figure 144.Strain data from Run 21: *(a)* B2575,*(b)* B2576,*(c)* B2918,*(d)* B2919,*(e)* B2920, and*(f)* B2921

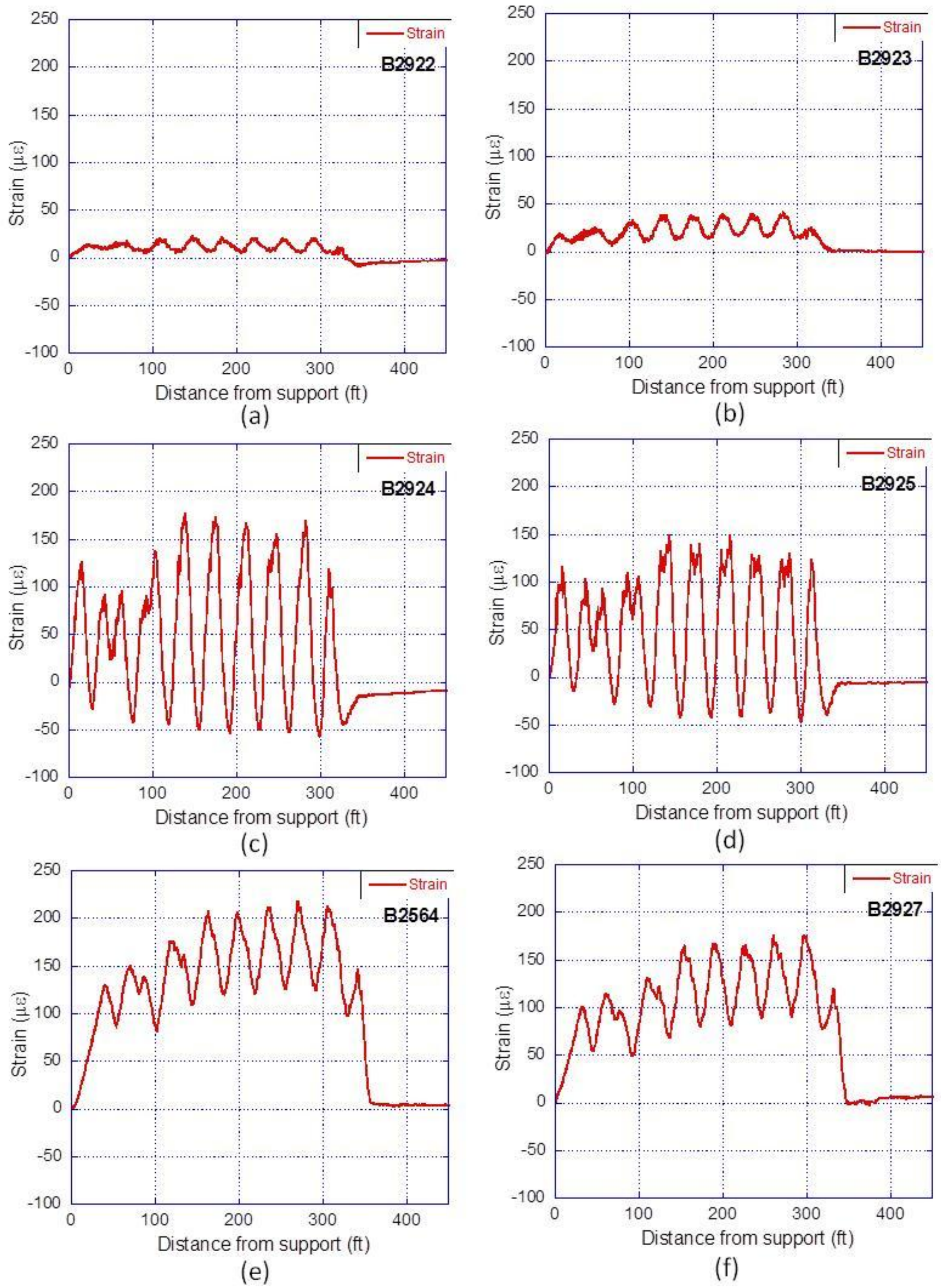


Figure 145.Strain data from Run 21: *(a)* B2922,*(b)* B2923,*(c)* B2924,*(d)* B2925,*(e)* B2564, and*(f)* B2927

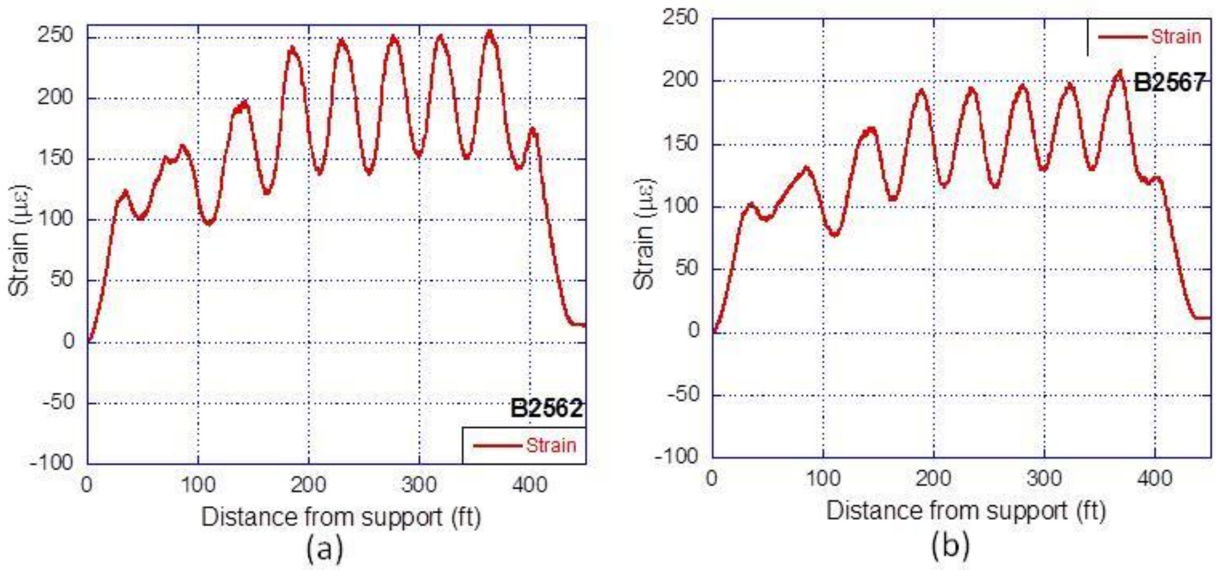


Figure 146.Strain data from Run 29: *(a)* B2562 and*(b)* B2567

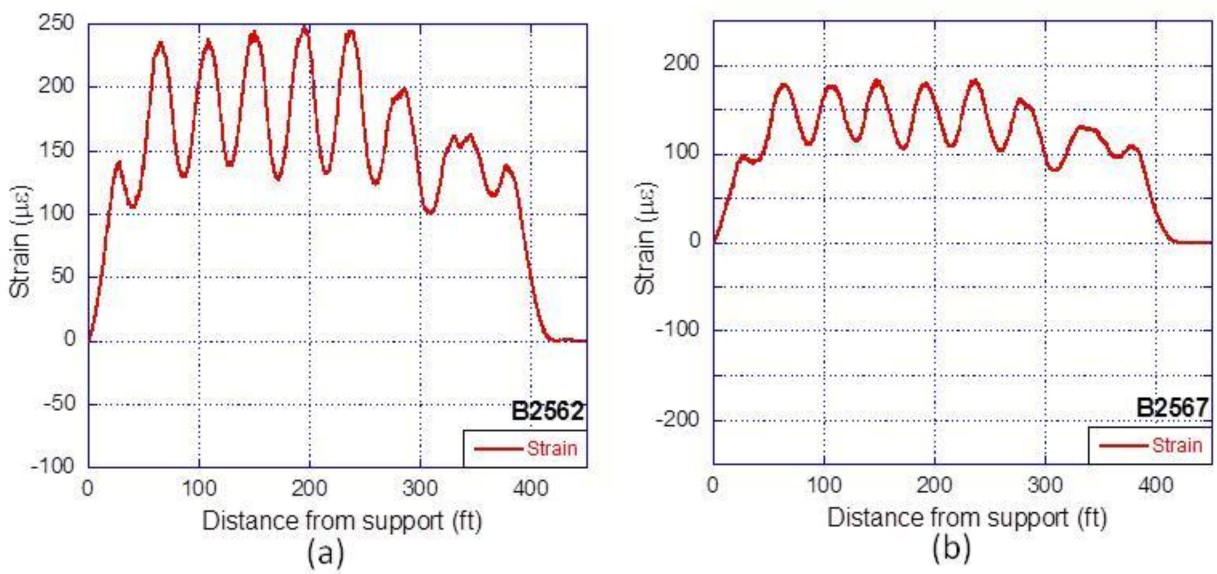


Figure 147.Strain data from Run 30: *(a)* B2562 and*(b)* B2567

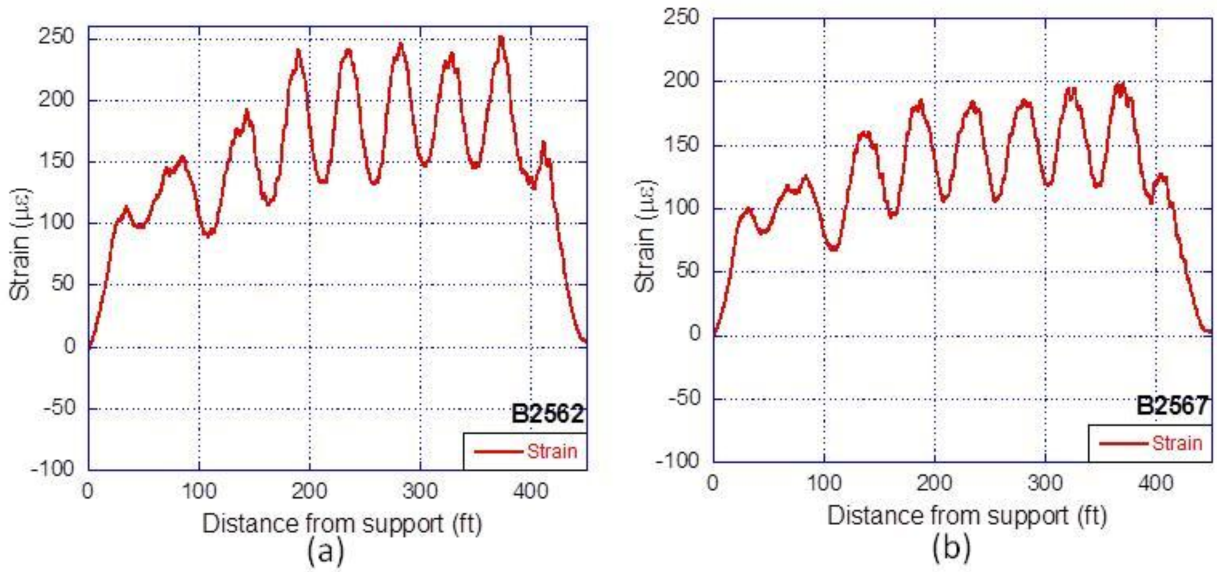


Figure 148.Strain data from Run 31: *(a)* B2562 and *(b)* B2567

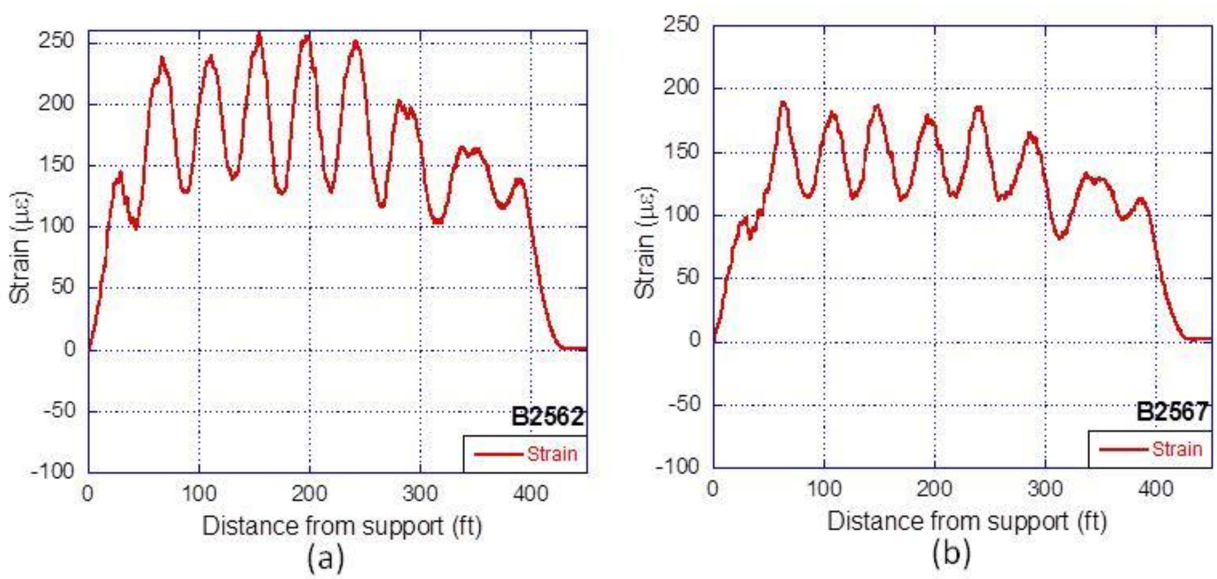


Figure 149.Strain data from Run 32: *(a)* B2562 and *(b)* B2567

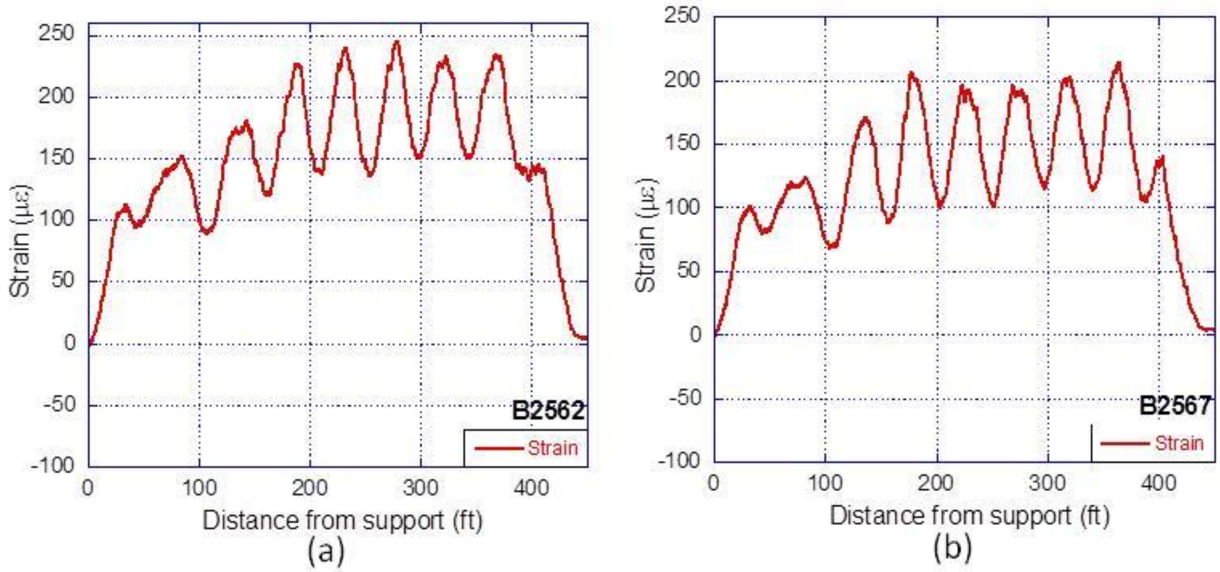


Figure 150.Strain data from Run 33: *(a)* B2562and *(b)* B2567

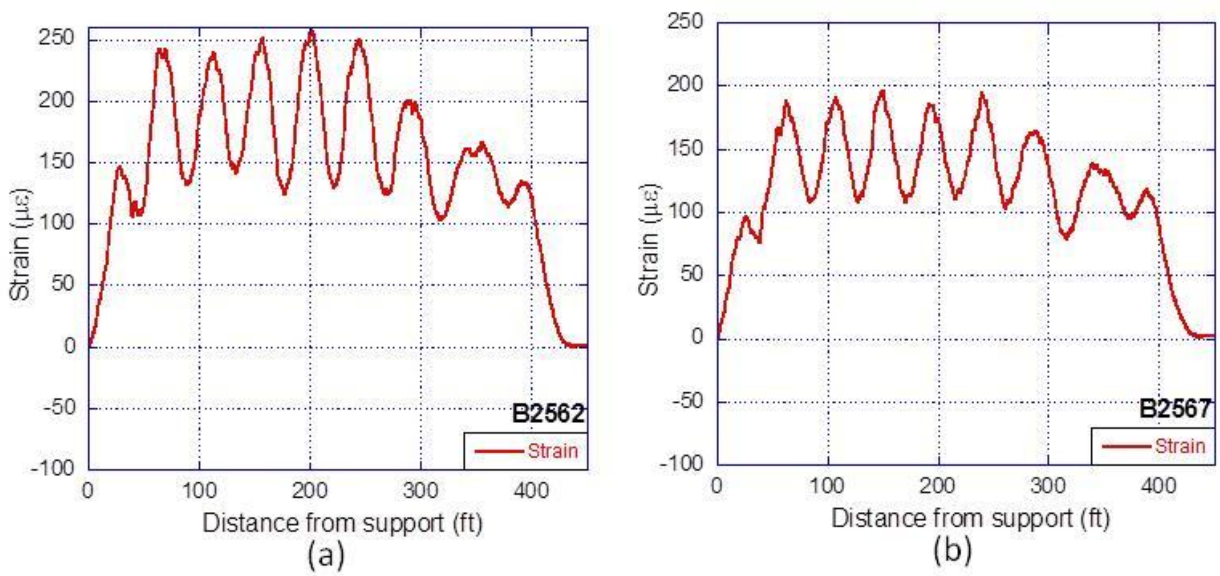


Figure 151.Strain data from Run 34: *(a)* B2562and *(b)* B2567

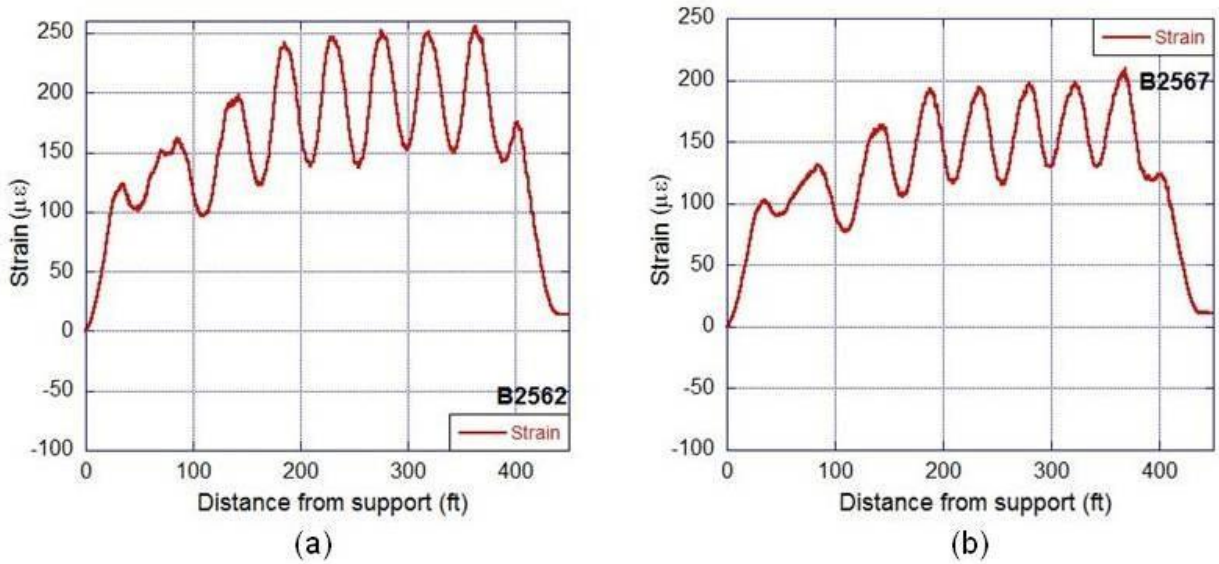


Figure 152.Strain data from Run 29 at 10mph speed:*(a)* B2562 and *(b)* B2567

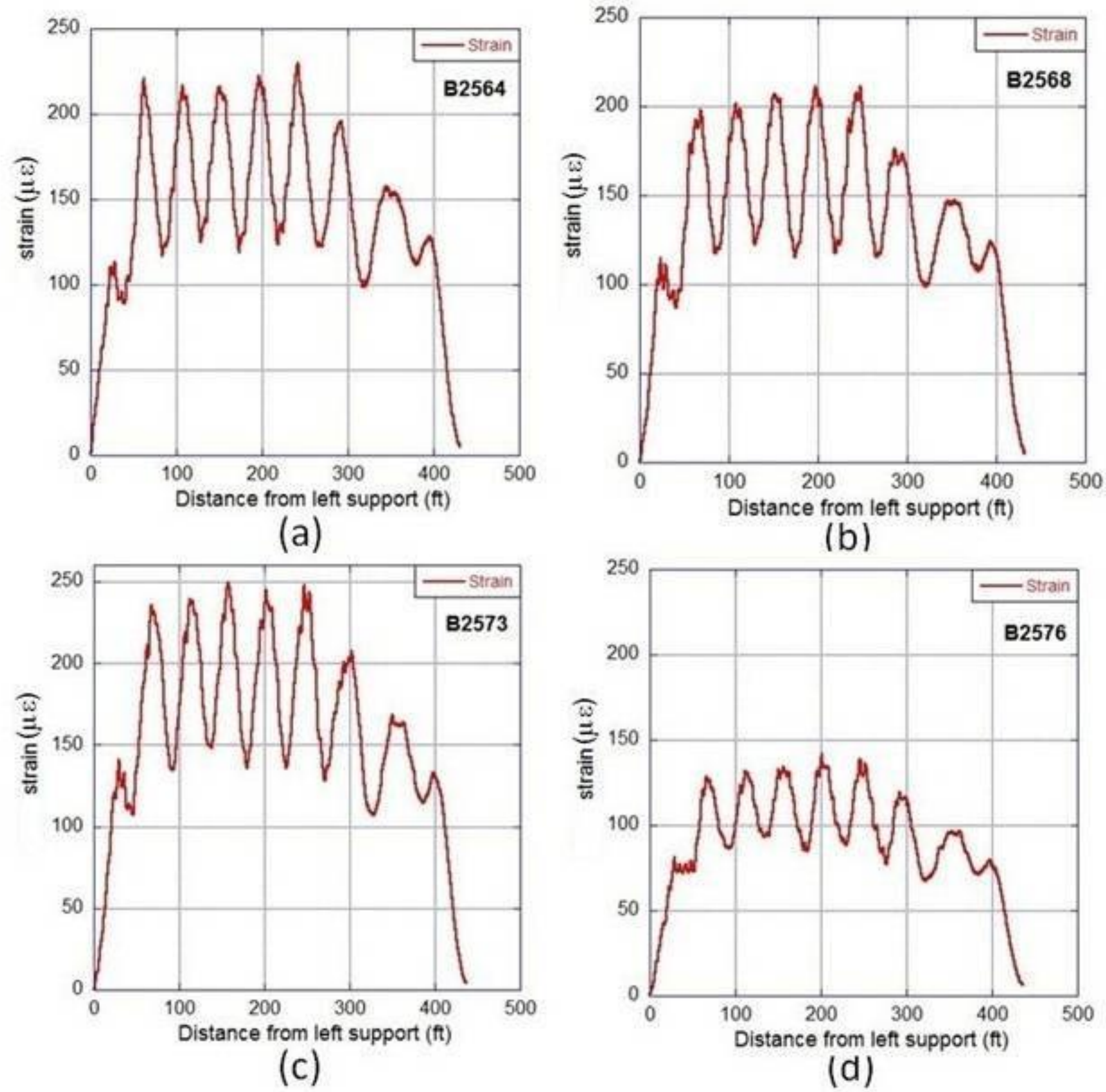


Figure 153. Strain data from Run 18 at 20 mph speed:*(a)* B2564,*(b)* B2568,*(c)* B2573, and*(d)* B2576

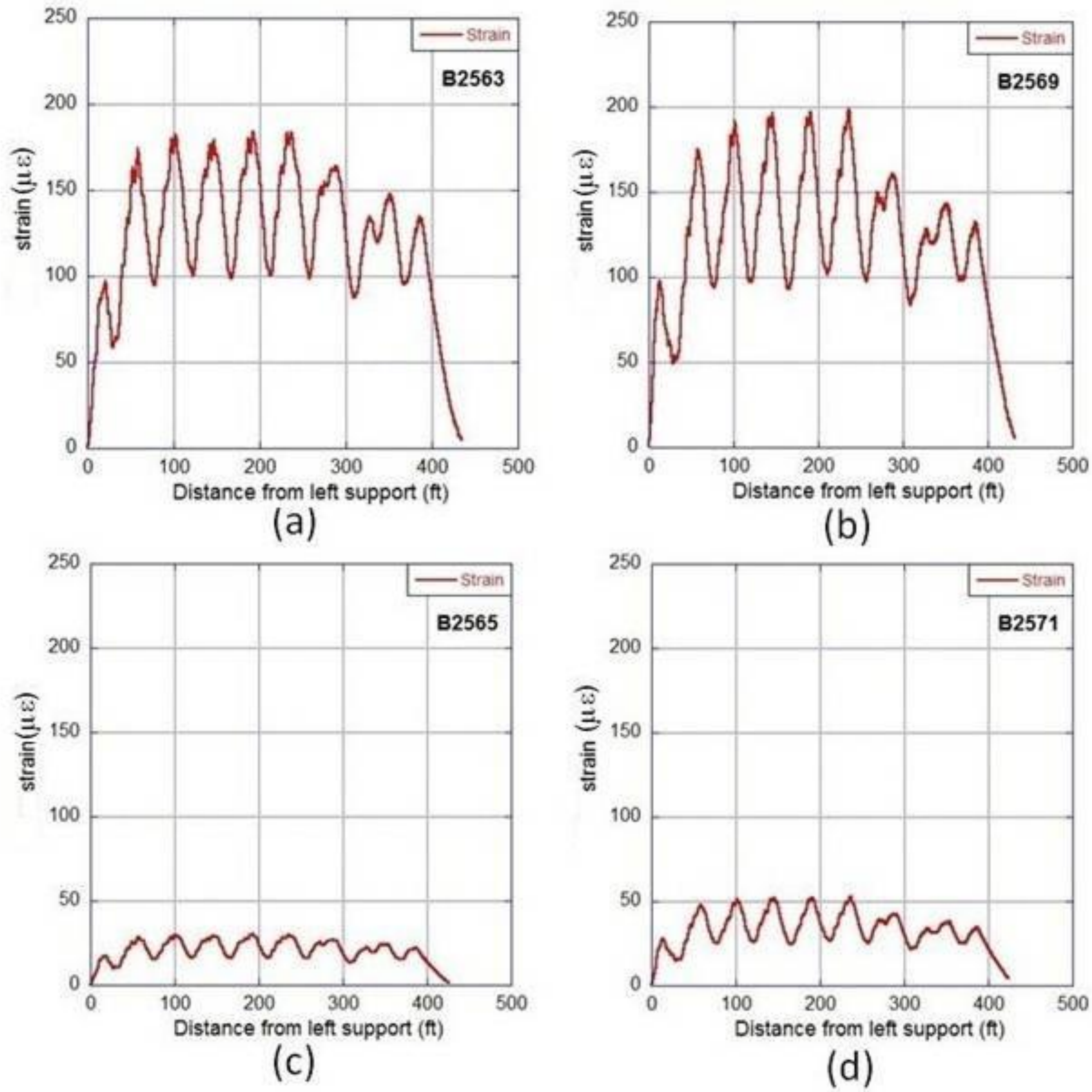


Figure 154. Strain data from Run 18 at 20 mph speed:*(a)* B2563,*(b)* B2569,*(c)* B2565, and*(d)* B2571

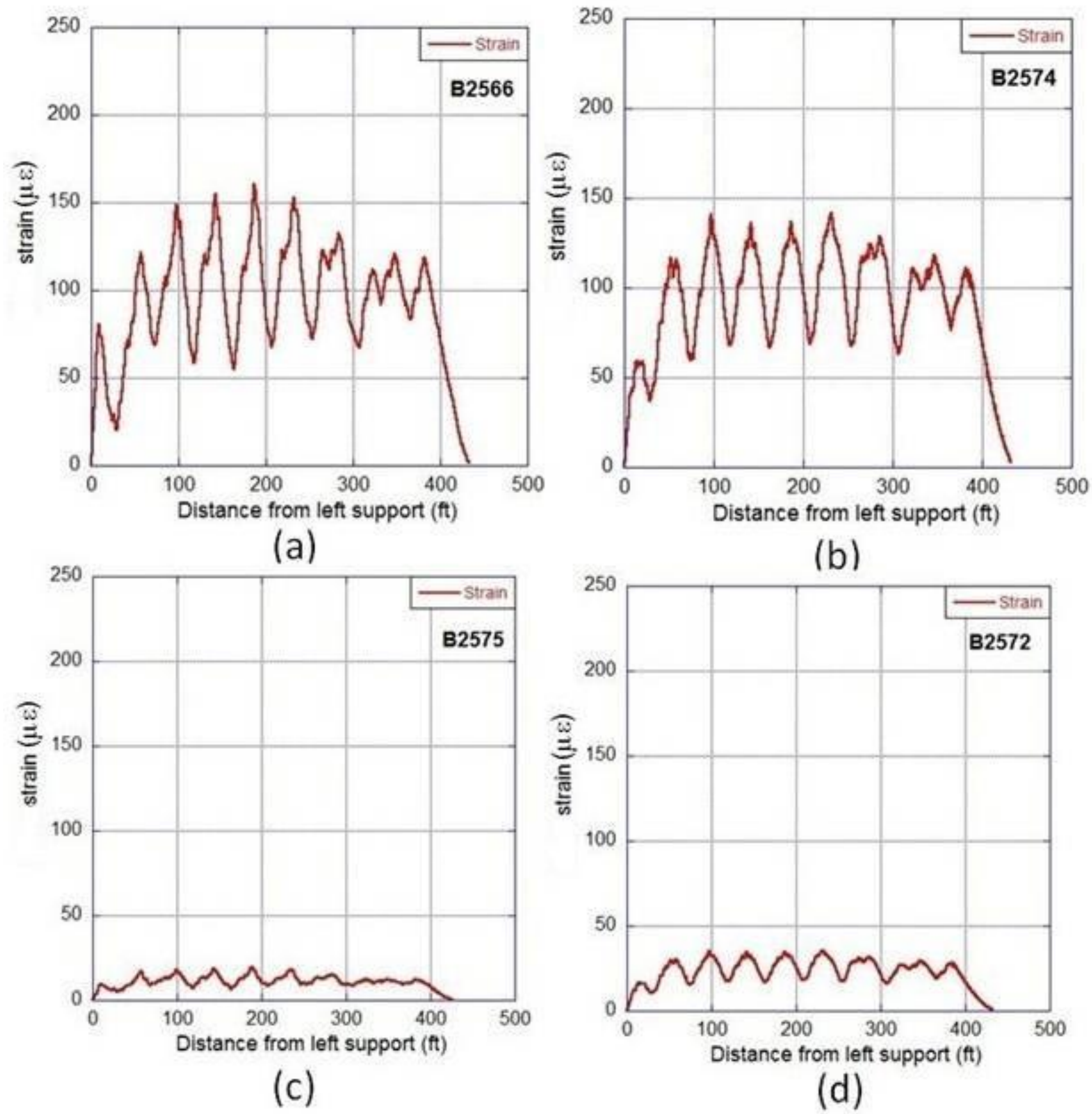


Figure 155. Strain data from Run 18 at 20 mph speed:*(a)* B2566,*(b)* B2574,*(c)* B2575, and *(d)* B2572

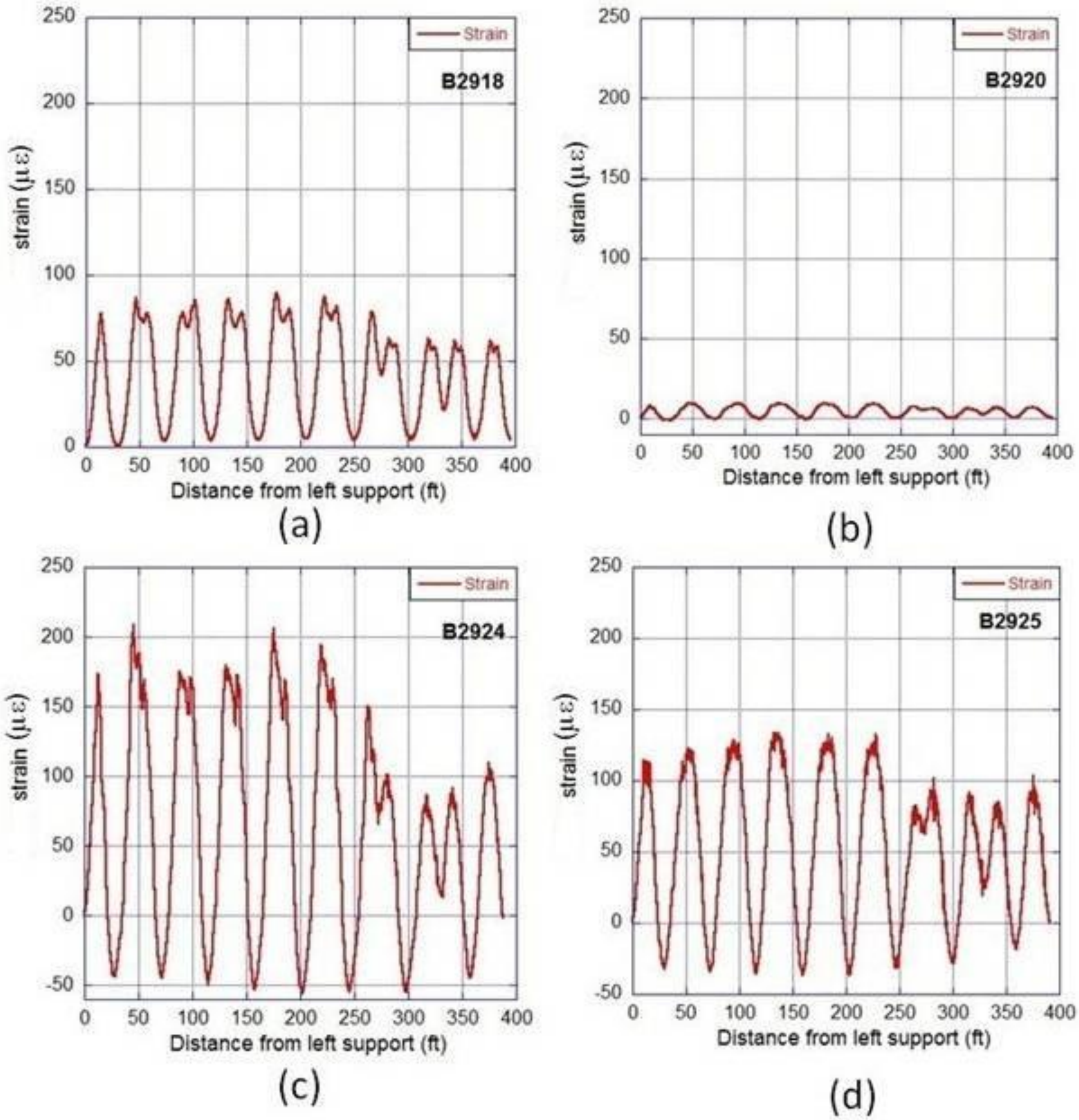


Figure 156. Strain data from Run 18 at 20 mph speed:*(a)* B2918,*(b)* B2920,*(c)* B2924, and*(d)* B2925

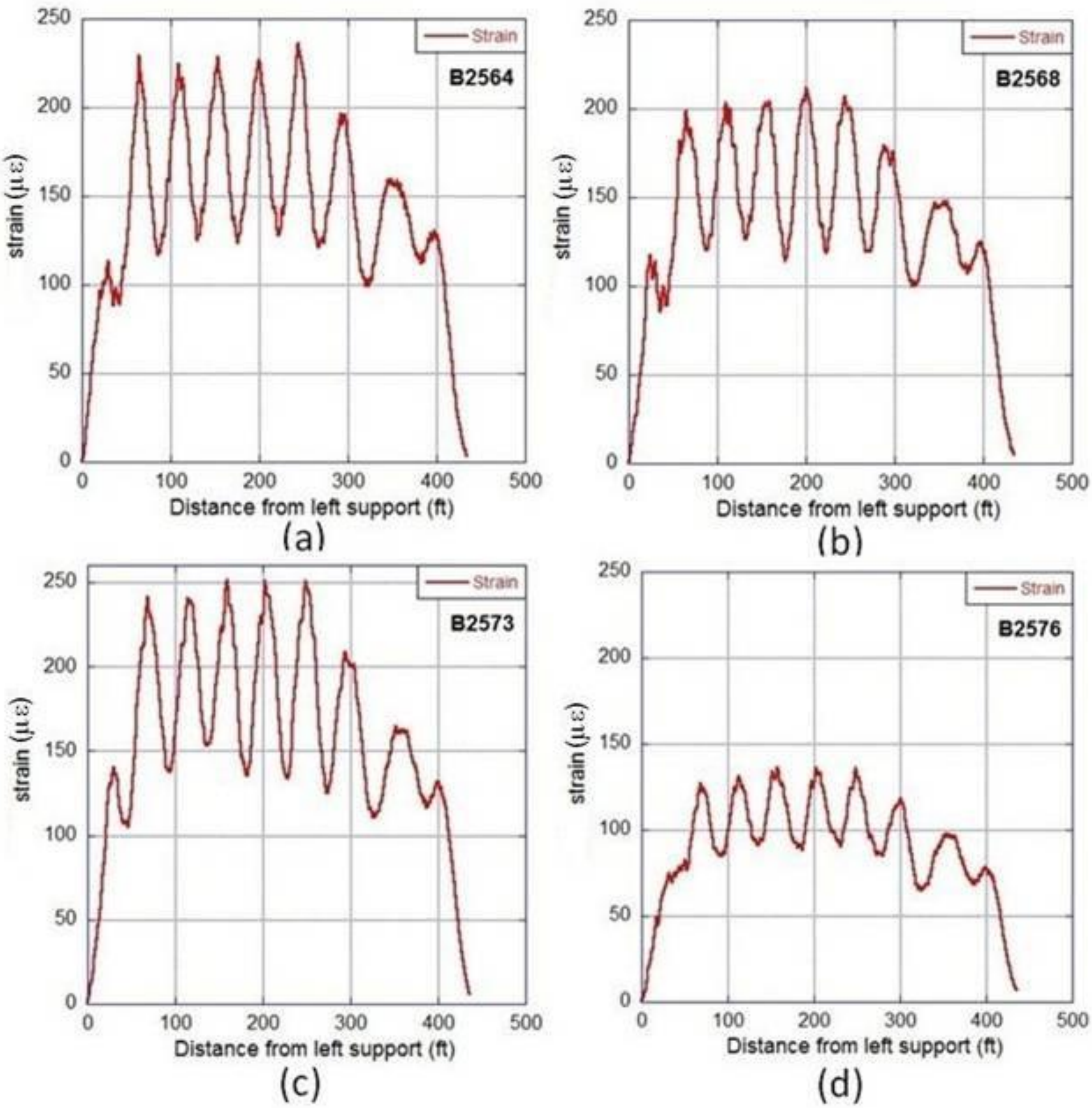


Figure 157. Strain data from Run 20 at 25 mph speed:*(a)* B2564,*(b)* B2568,*(c)* B2573, and*(d)* B2576

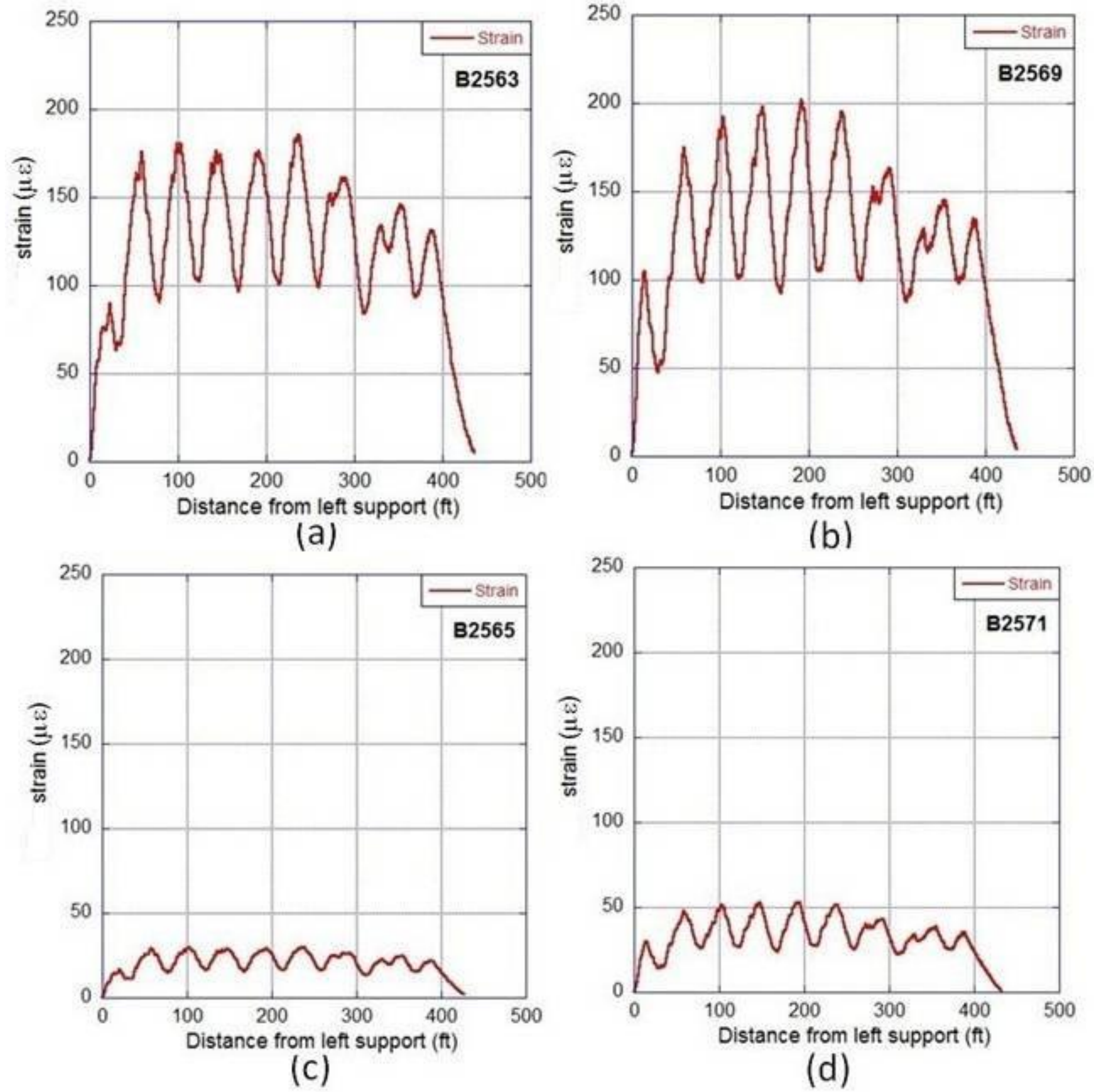


Figure 158. Strain data from Run 20 at 25 mph speed *(a)* B2563,*(b)* B2569,*(c)* B2565, and*(d)* B2571

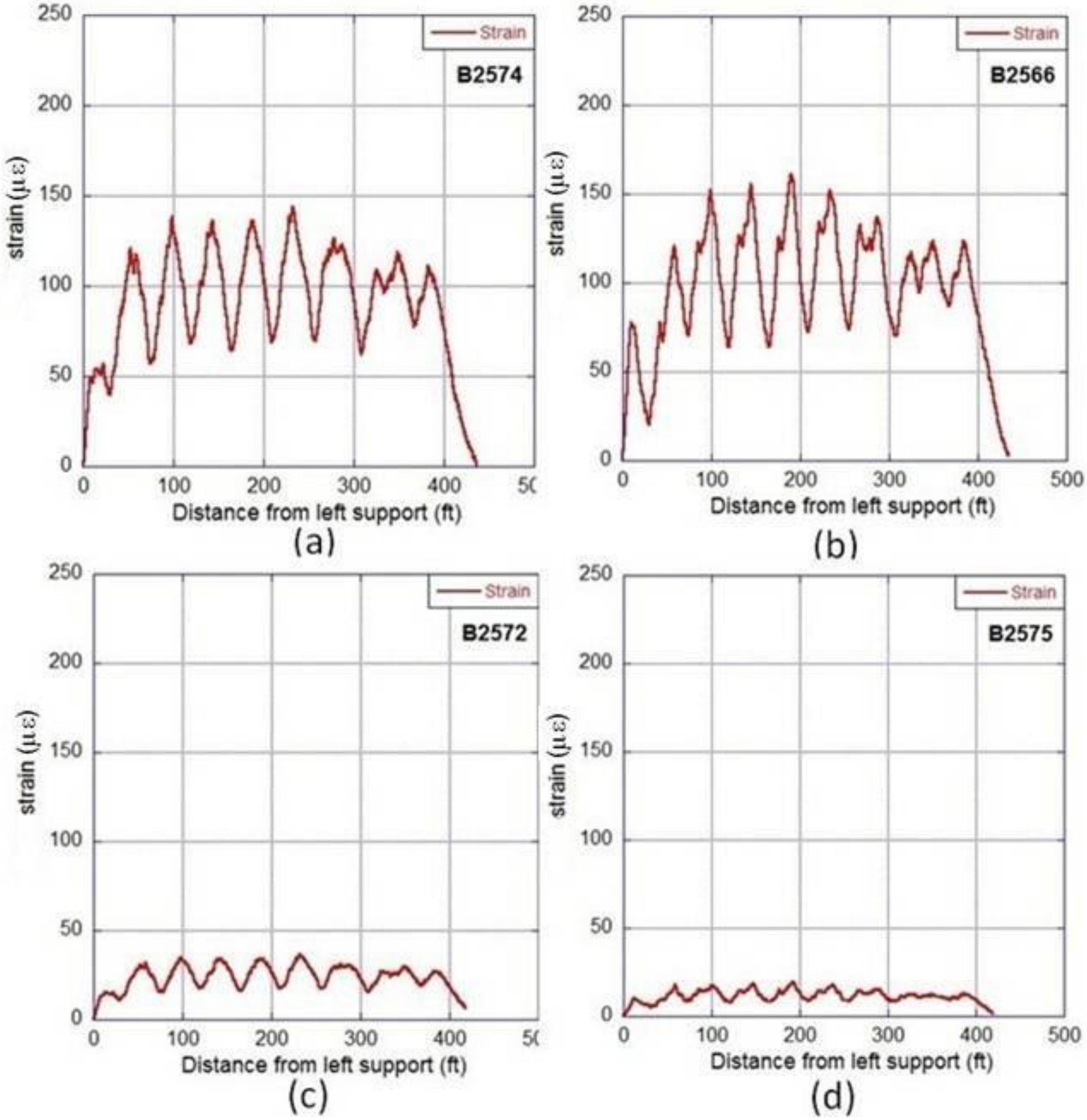


Figure 159. Strain data from Run 20 at 25 mph speed:*(a)* B2574,*(b)* B2566,*(c)* B2572, and*(d)* B2975

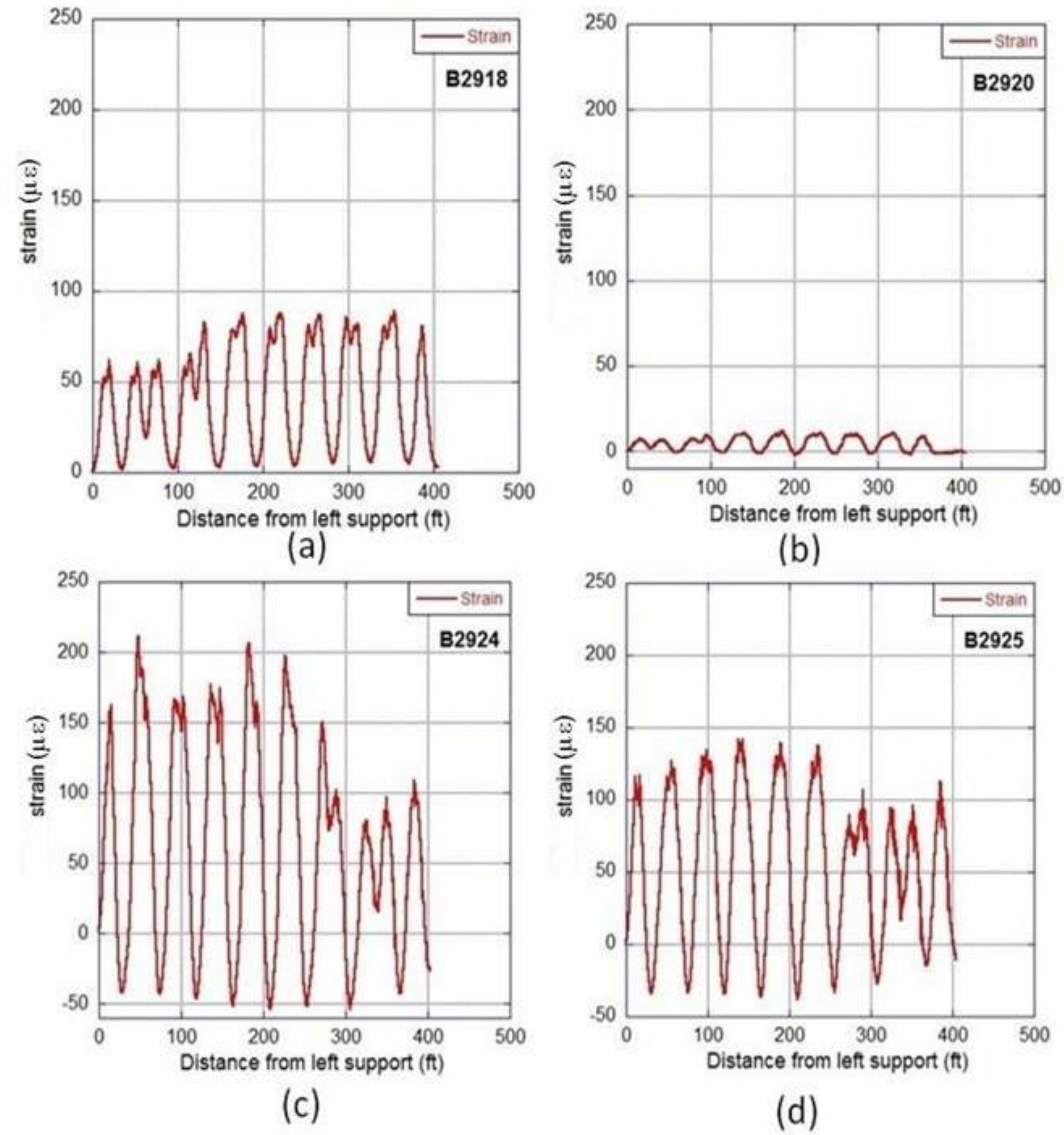


Figure 160. Strain data from Run 20 at 25 mph speed:*(a)* B2918,*(b)* B2920,*(c)* B2924, and*(d)* B2925

APPENDIX D: EXPERIMENTAL DATA COLLECTED FROM RARITAN VALLEY LINE MP 31.15 (MIDDLE BROOK) BRIDGE

Table 77 and Table 78 present the information collected from each experimental run, and Figure 163through Figure 171 shows the experimental data collected from the field.

Table 77 - Information about the test trains on 09/23/2011 for Raritan Valley Line MP 31.15 (Middle Brook)

	Information About the Tested Train			
	Direction	**Time**	**Speed (mph)**	**Track #**
Run #1	Bound Brook–Bridgewater	9/22/2011 15:55	34.2	2
Run #2	Bridgewater–Bound Brook	9/22/2011 16:00	N/A	1
Run #3	Bound Brook–Bridgewater	9/22/2011 16:25	N/A	2
Run #4	Bridgewater–Bound Brook	9/23/2011 10:10	36.0	1
Run #5	Bound Brook–Bridgewater	9/23/2011 11:03	35.5	1
Run #6	Bridgewater–Bound Brook	9/23/2011 11:29	37.0	1

Table 78 - Information about the test trains on 09/30/2011 for Raritan Valley Line MP 31.15 (Middle Brook)

Run #	**Arrival Time**	**Direction**	**Track**	**Train #**	**Laser**	**File Name**
7	8:44 a.m.	Bound brook–Bridgewater (W)	1	5413	N/A	0923-2
8	8:52 a.m.	Bridgewater–Bound Brook (E)	2	5426	N/A	0923-3
9	9:49 a.m.	Bound Brook–Bridge water (W)	1	5415	G8	0923-4
10	9:56 a.m.	Bridgewater–Bound Brook (E)	2	5730	G8	0923-5
11	10:51 a.m.	Bound Brook–Bridgewater (W)	1	5719	G8	0923-6
12	10:56 a.m.	Bridgewater–Bound Brook (E)	2	5432	G6	0923-7

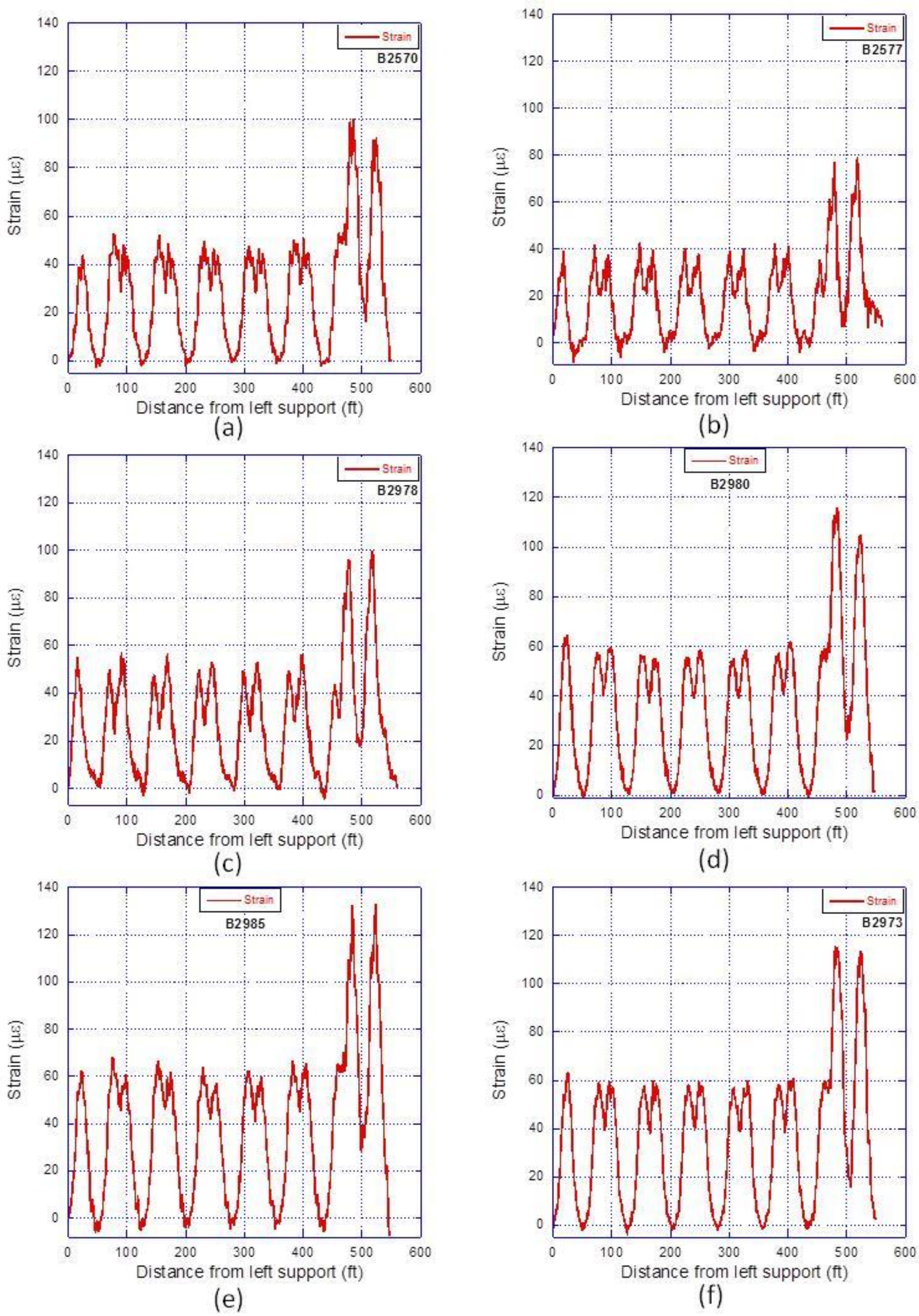


Figure 161.Strain data from Run 3: *(a)* B2570,*(b)* B2577,*(c)* B2978,*(d)* B2980,*(e)* B2985, and*(f)* B2973

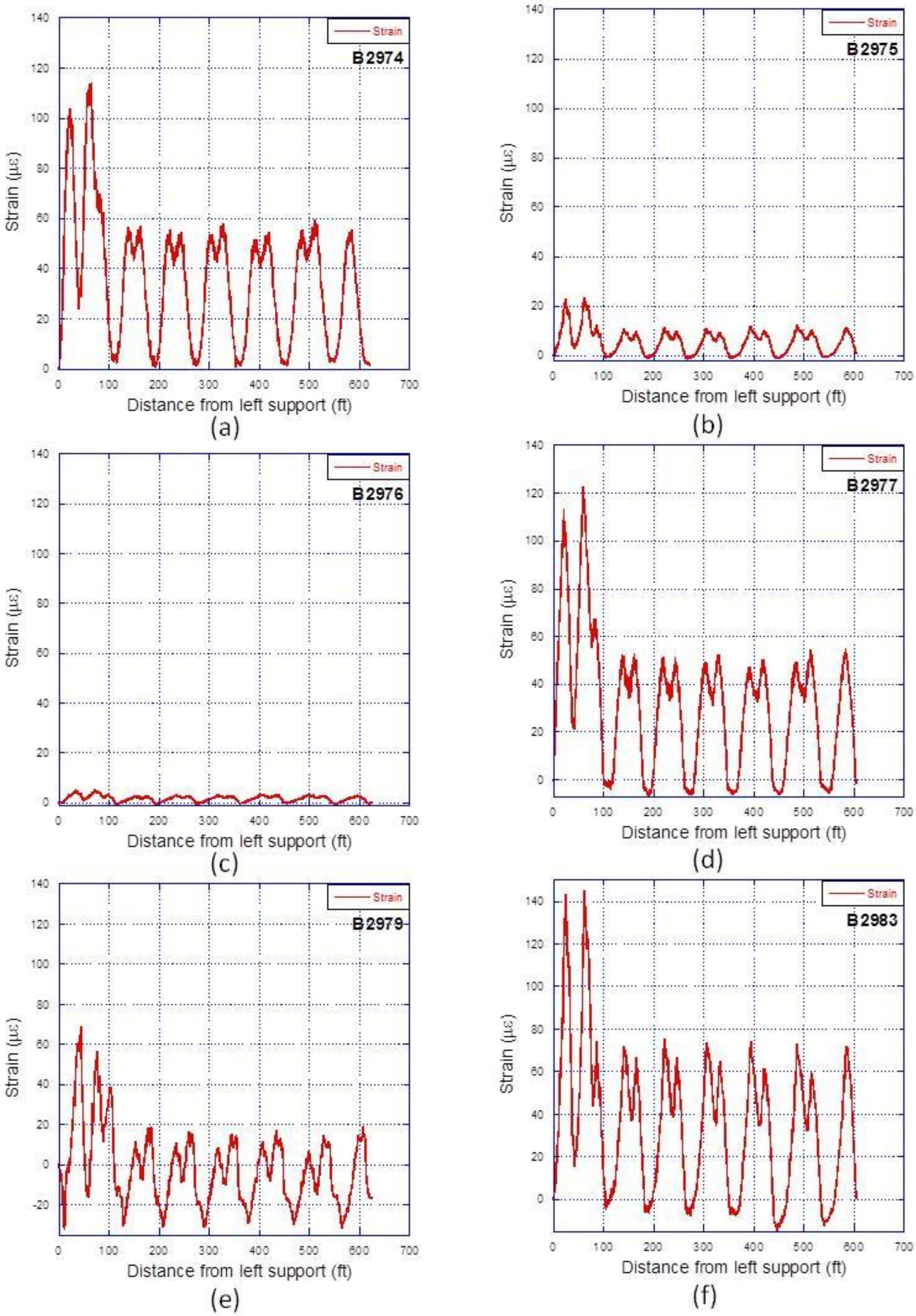


Figure 162.Strain data from Run 6: *(a)* B2974;*(b)* B2975,*(c)* B2976,*(d)* B2977,*(e)* B2979, and*(f)* B2983

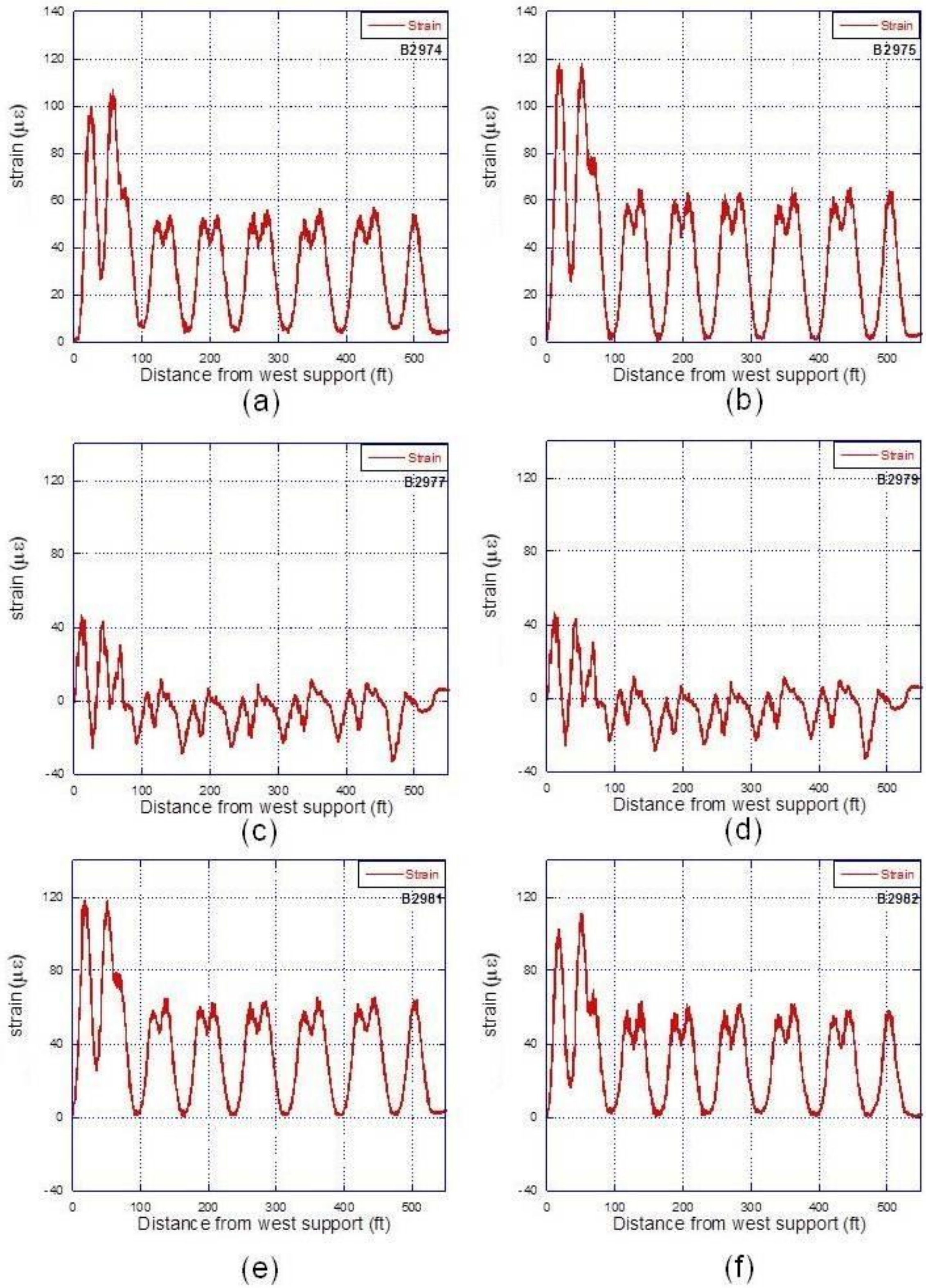


Figure 163.Strain data from Run 7: *(a)* B2974,*(b)* B2975,*(c)* B2977,*(d)* B2979,*(e)* B2981, and*(f)* B2982

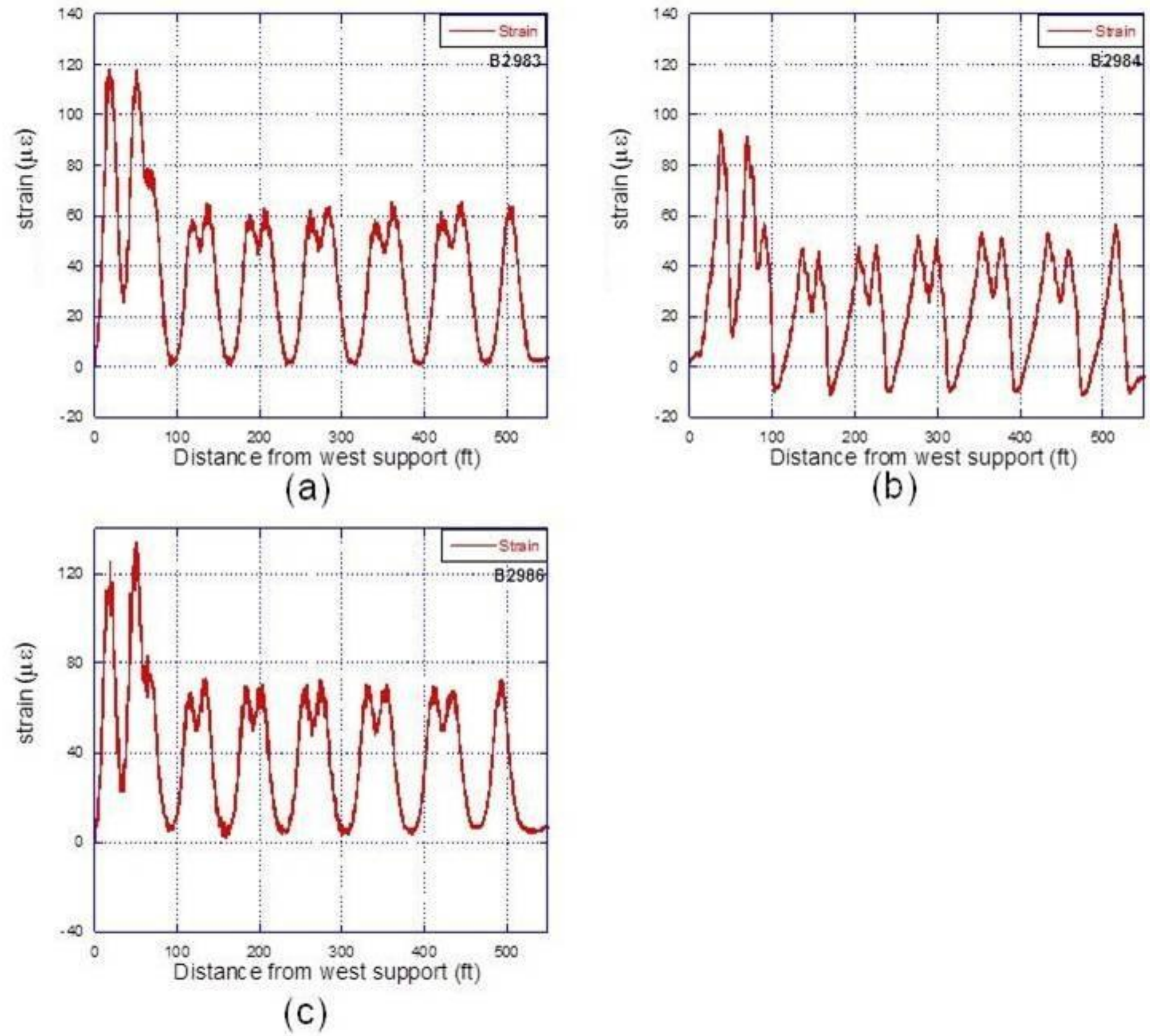


Figure 164.Strain data from Run 7: *(a)* B2983,*(b)* B2984, and*(c)* B2986

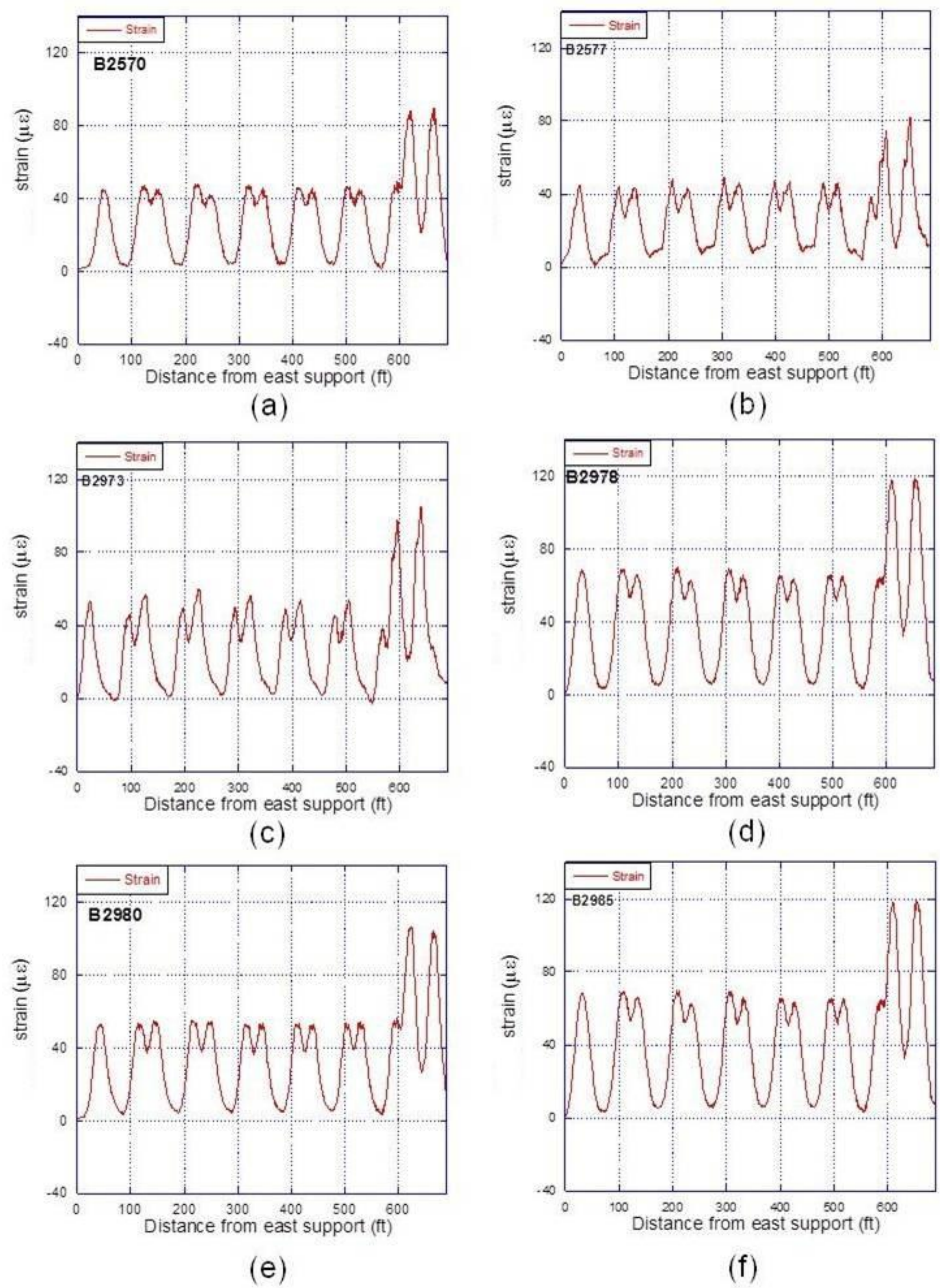


Figure 165.Strain data from Run 8: *(a)* B2570,*(b)* B2577,*(c)* B2973,*(d)* B2978,*(e)* B2980, and*(f)* B2985

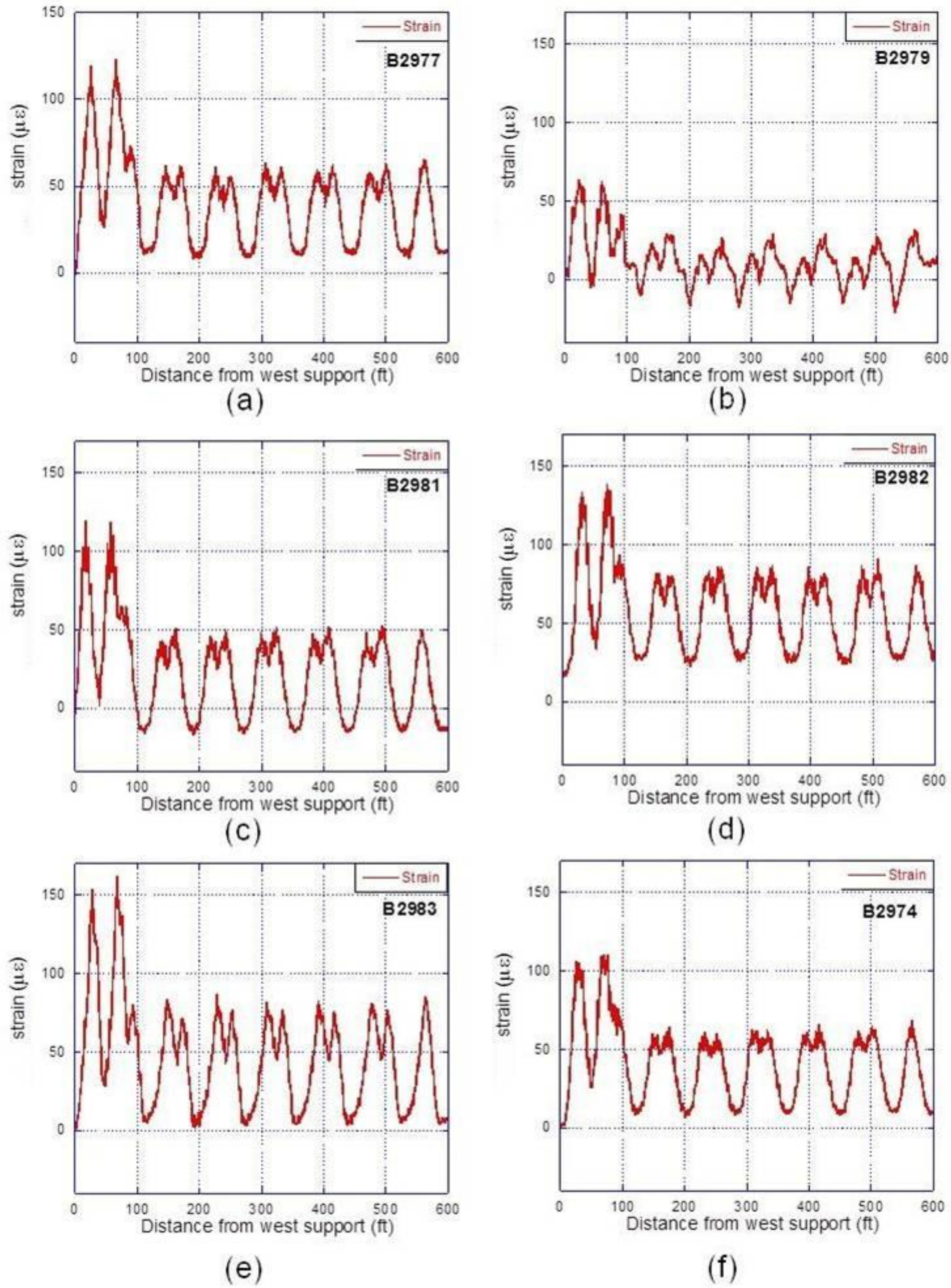


Figure 166.Strain data from Run 9: *(a)* B2977,*(b)* B2979,*(c)* B2981,*(d)* B2982,*(e)* B2983, and*(f)* B2974

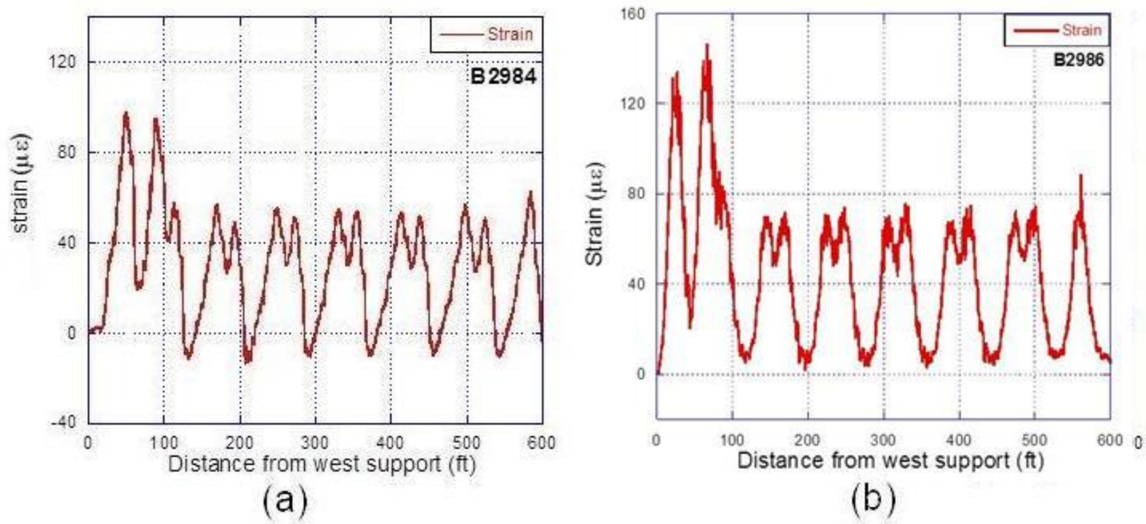


Figure 167.Strain data from Run 9: *(a)* B2984 and*(b)* B2986

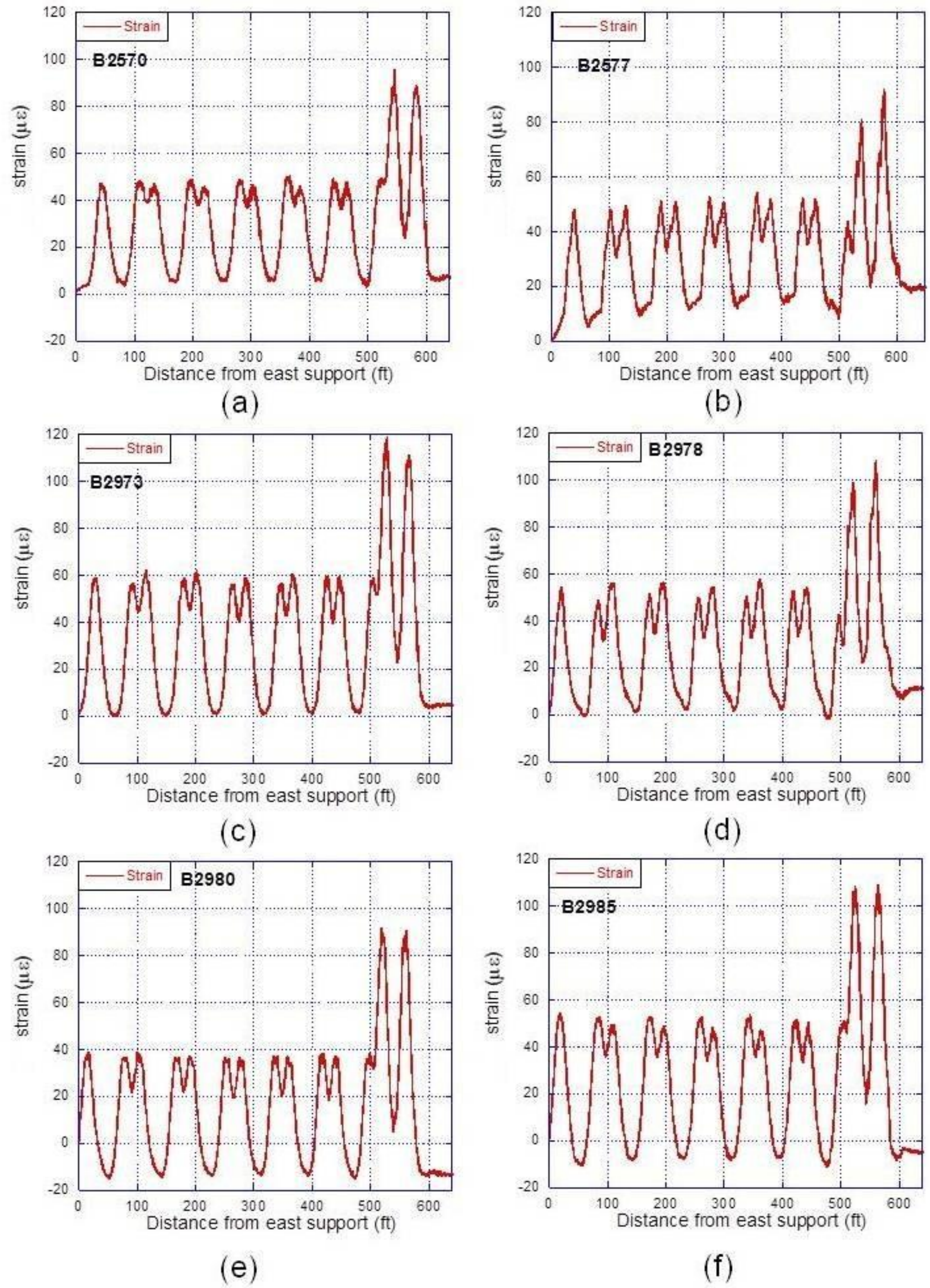


Figure 168.Strain data from Run 10: *(a)* B2570,*(b)* B2577,*(c)* B2973,*(d)* B2978,*(e)* B2980, and*(f)* B2985

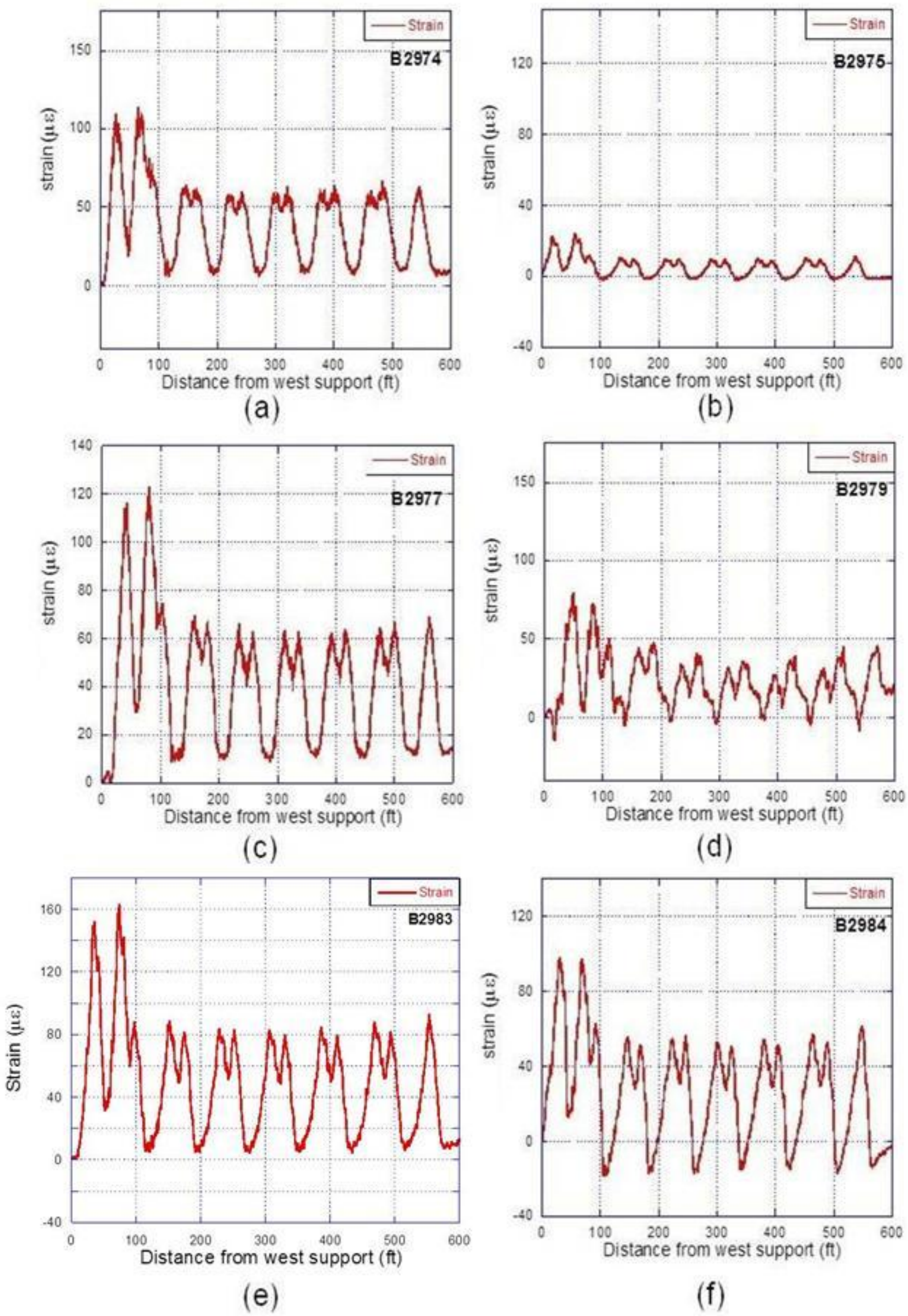


Figure 169.Strain data from Run 11: *(a)* B2974,*(b)* B2975,*(c)* B2977,*(d)* B2979,*(e)* B2983, and*(f)* B2984

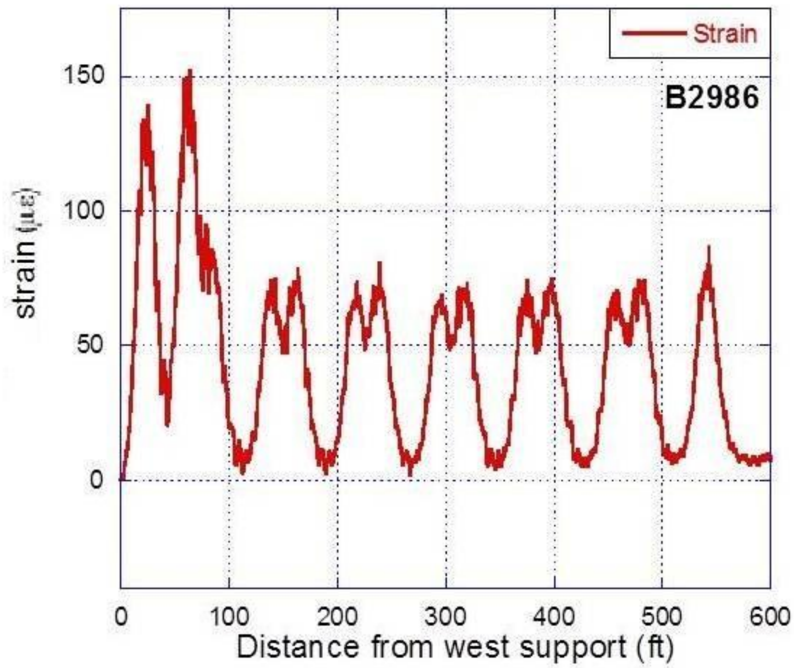


Figure 170.Strain data from Run 11: B2986

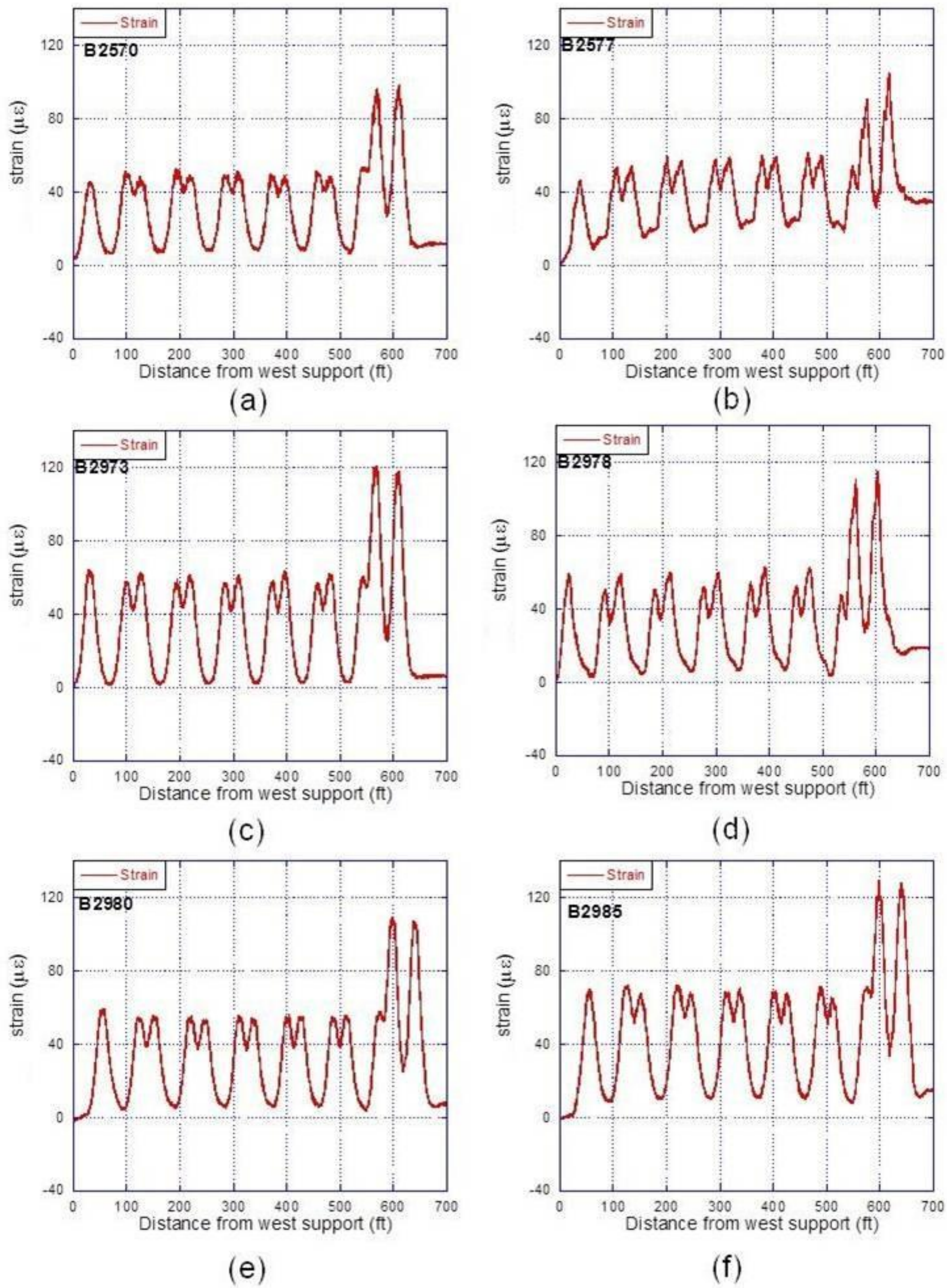


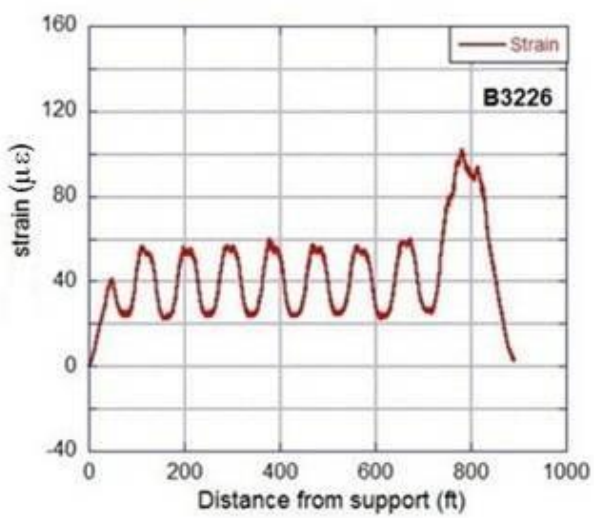
Figure 171.Strain data from Run 12: *(a)* B2570,*(b)* B2577,*(c)* B2973,*(d)* B2978,*(e)* B2980, and*(f)* B2985

APPENDIX E: EXPERIMENTAL DATA COLLECTED FROM NORTH JERSEY COAST LINE MP 0.39 (RIVERDRAW) BRIDGE

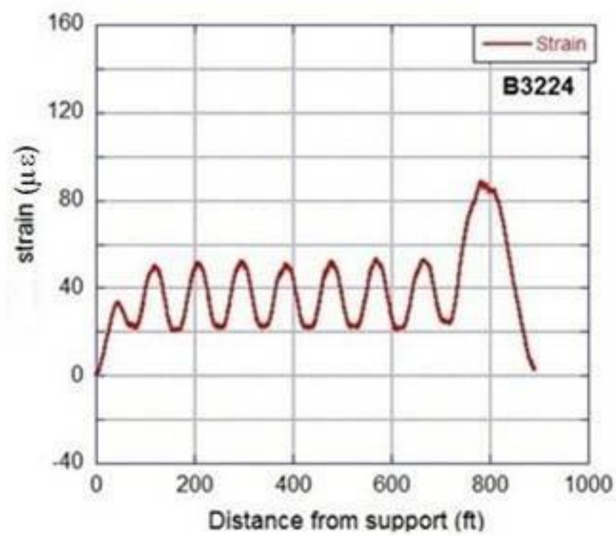
Table 79 presents the information collected from each experimental case, and Figure 180 through Figure 187 show the experimental data collected from the field.

Table 79 - Information about the test trains for North Jersey Coast Line MP 0.39 (River Draw)

Run #	Arrival Time	Direction	Track	Train Symbol	Number of Cars	Locomotive Type	Speed
1	9:38 a.m.	Westbound	1	3227	9	ALP-46A	15.3
2	10:42 a.m.	Westbound	1	3231	8	ALP-46A	17.6
3	12:34 p.m.	Eastbound	2	3244	8	ALP-46A	24.2
4	12:42 p.m.	Westbound	1	3239	9	ALP-46A	12.4
5	1:34 p.m.	Eastbound	2	3248	9	ALP-46	31.7
6	1:40 p.m.	Westbound	1	3243	6	ALP-46A	18.3

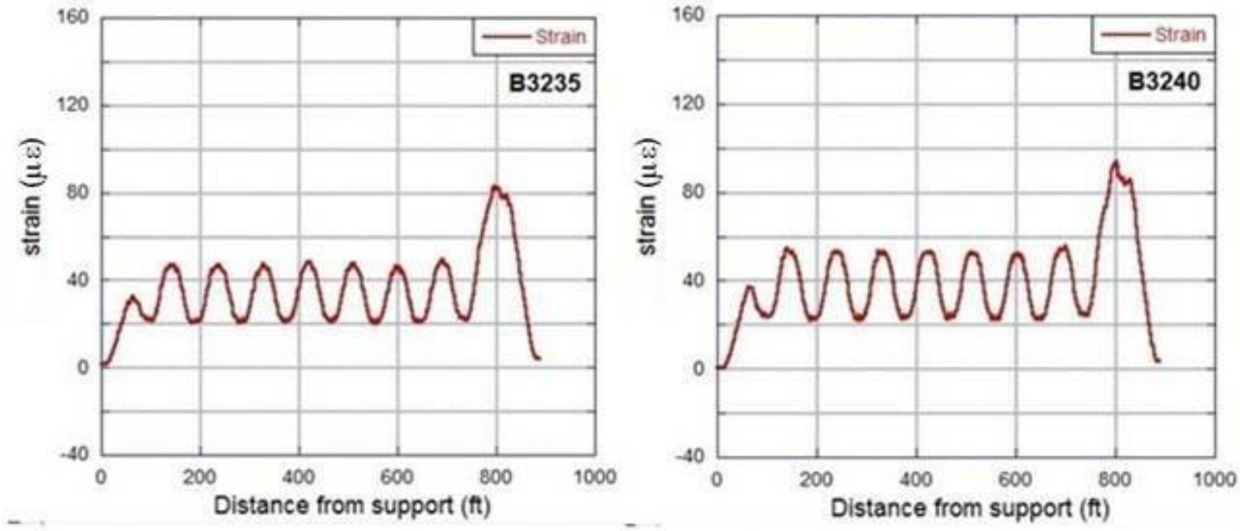


(a)



(b)

Figure 172.Strain data measured during Test Run #1: *(a)* B3226 and*(b)* B3224



(a) *(b)*

Figure 173.Strain data measured during Test Run #5: *(a)* B3235 and*(b)* B3240

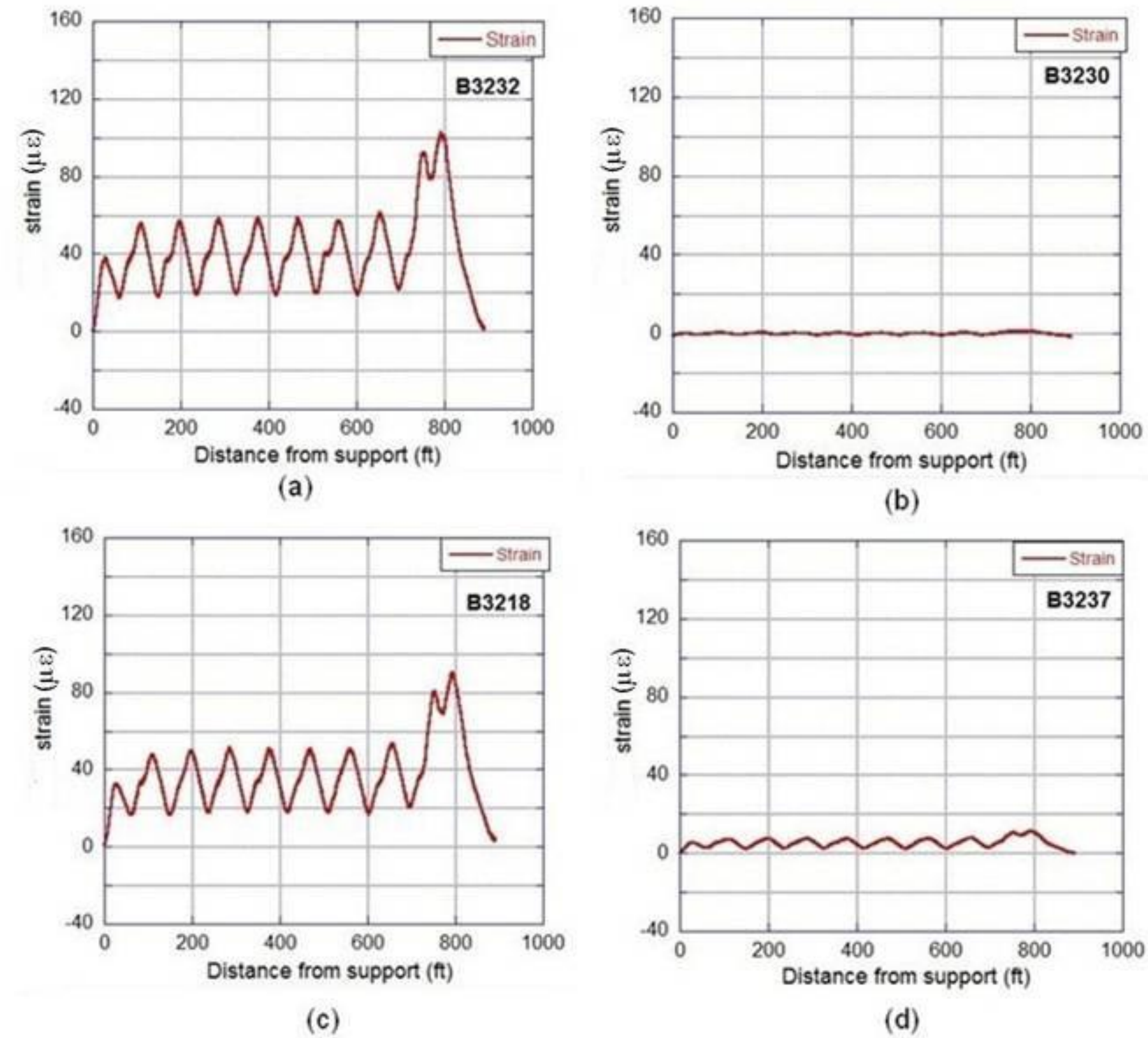


Figure 174.Strain data measured during test run#1: *(a)* B3232, *(b)* 3230, *(c)* B3218, and*(d)* B3237

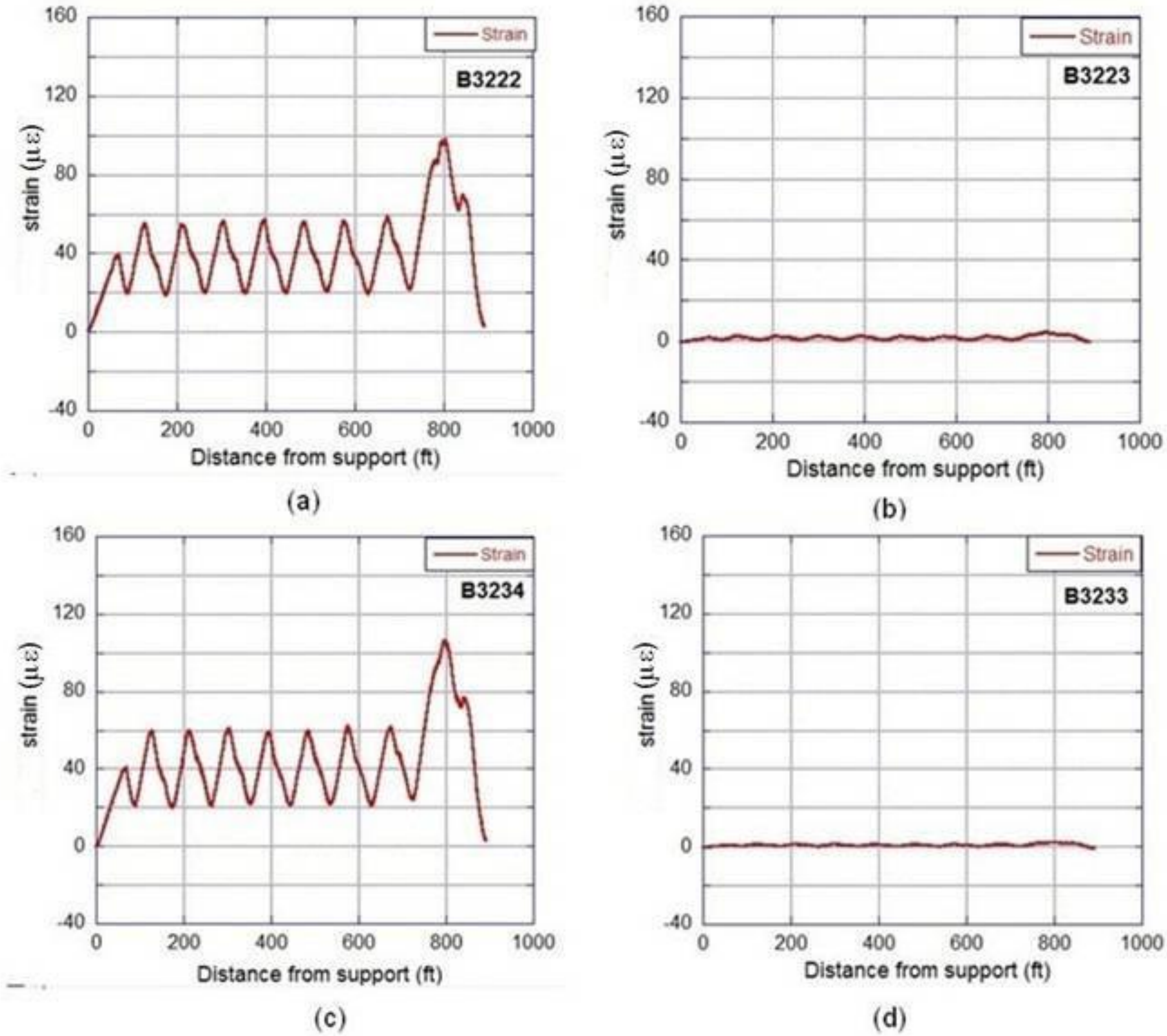


Figure 175.Strain data measured during test run #1:*(a)* B3222, *(b)* B3223, *(c)* B3234, and *(d)* B3233

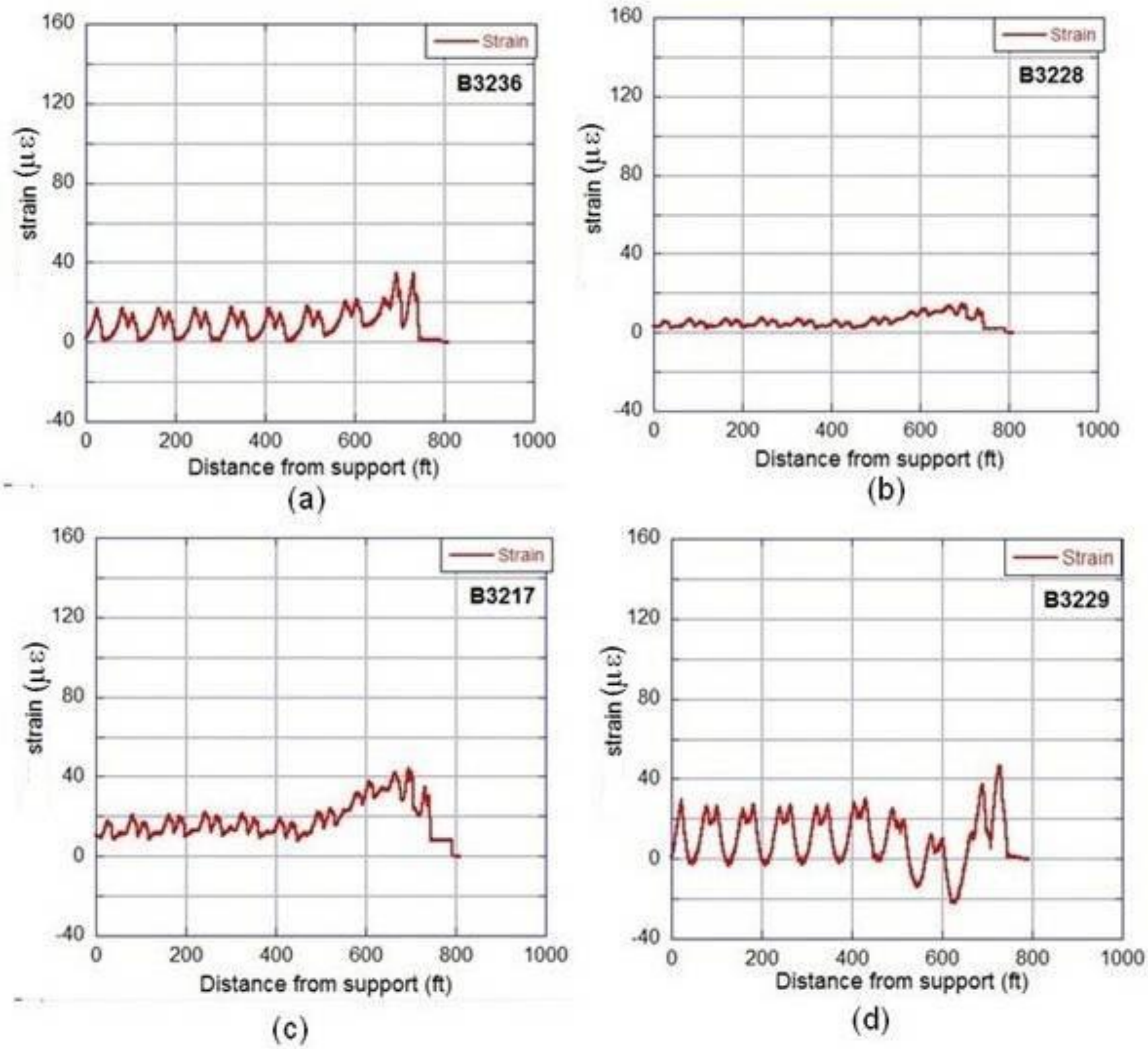


Figure 176.Strain data measured during test run #1:*(a)* B3236, *(b)* B3228, *(c)* B3217, and (d) B3229

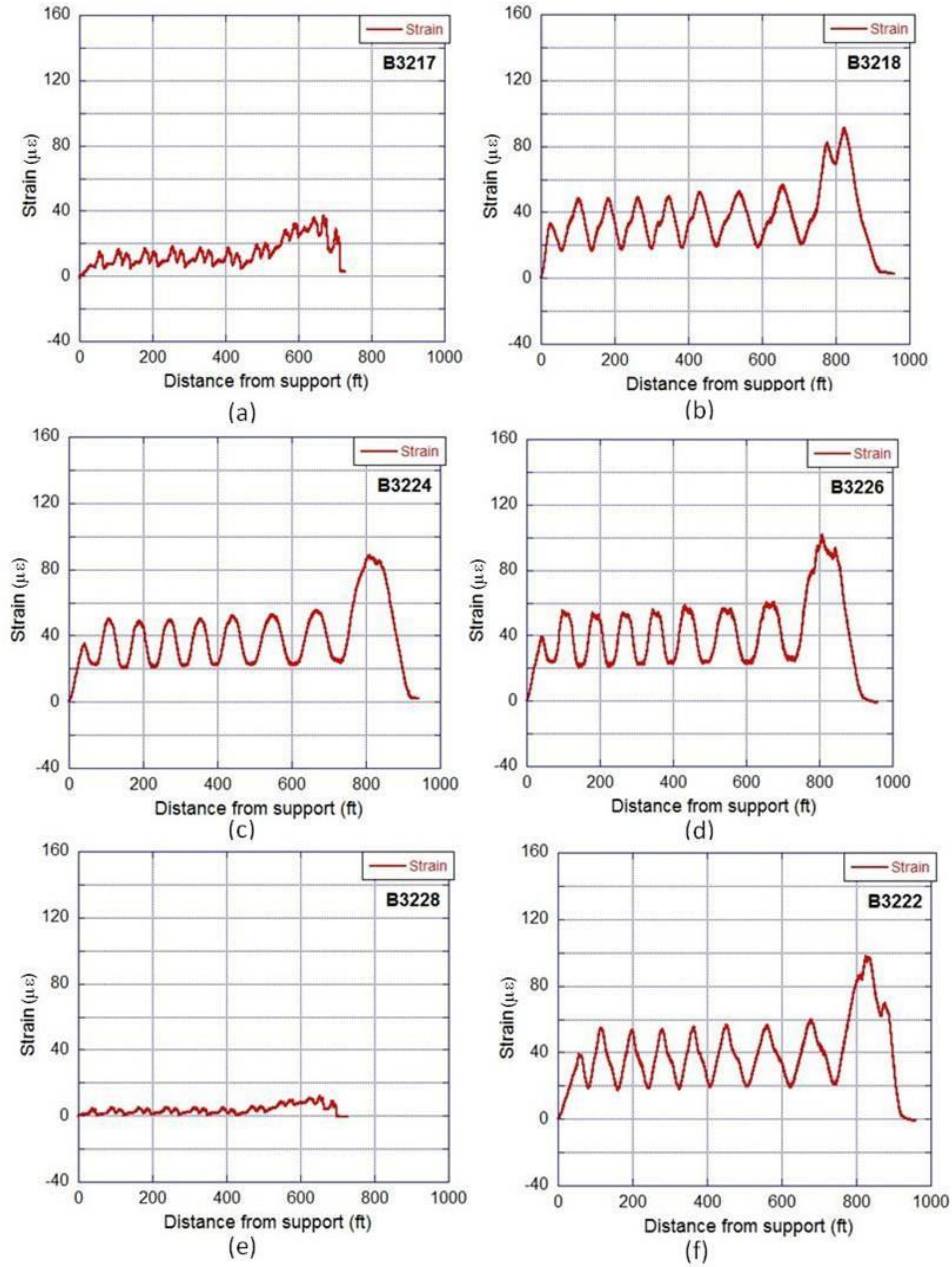


Figure 177.Strain data from Run 2: *(a)* B3217,*(b)* B3218,*(c)* B3224,*(d)* B3226,*(e)* B3228, and*(f)* B3222

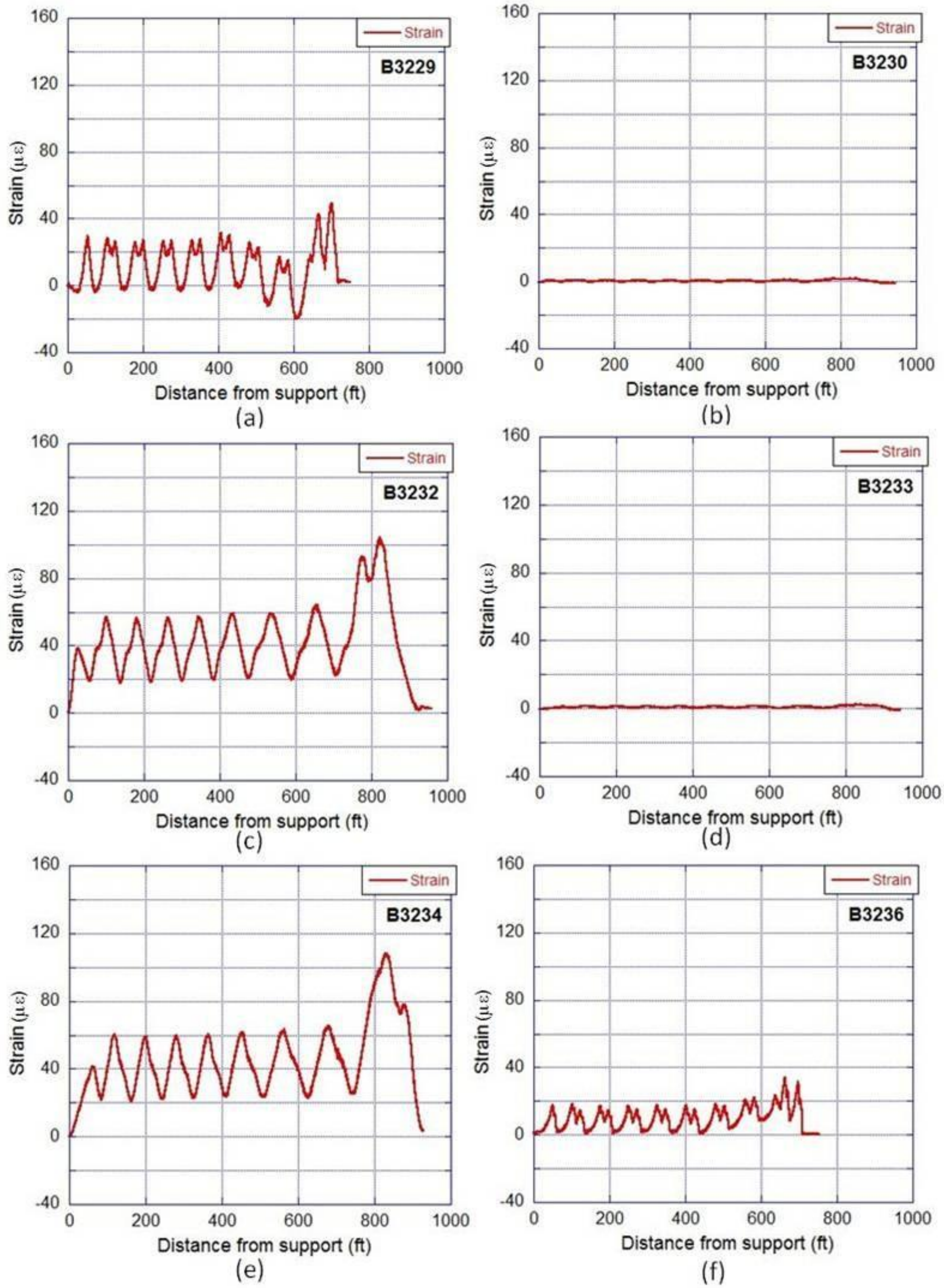


Figure 178.Strain data from Run 2: *(a)* B3229,*(b)* B3230,*(c)* B3232,*(d)* B3233,*(e)* B3234, and*(f)* B3236

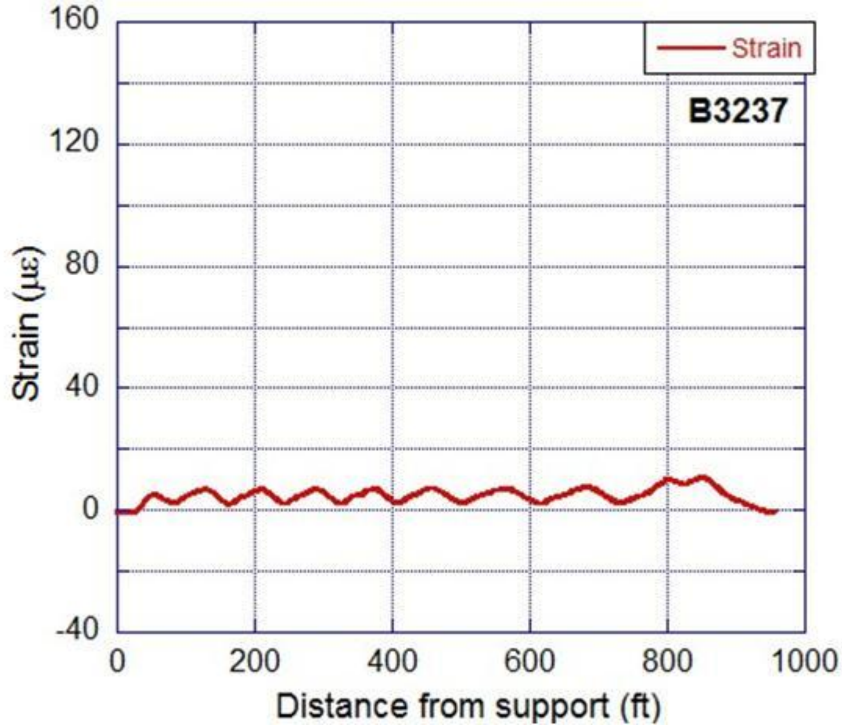


Figure 179.Strain data from Run 2: B3237

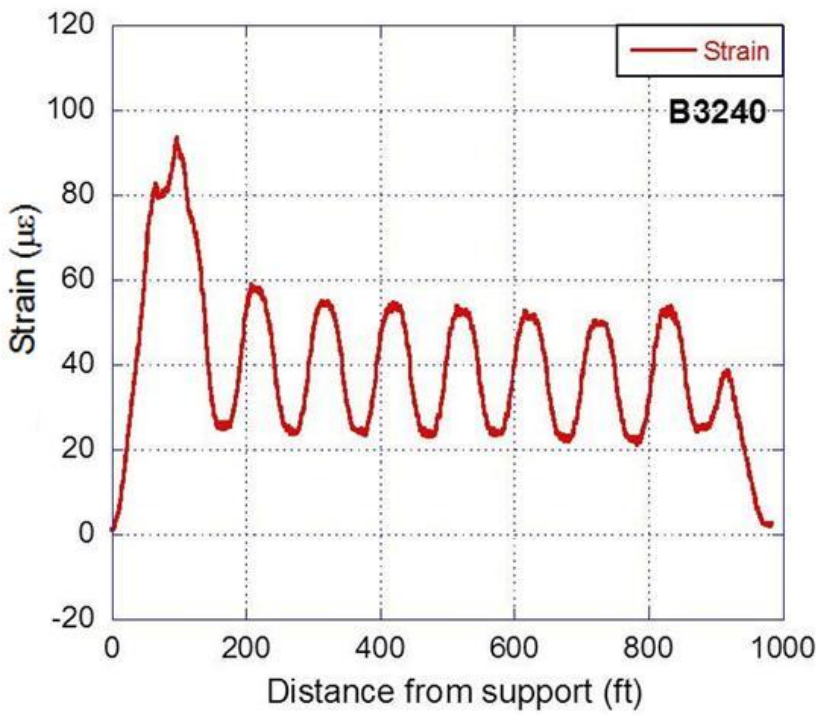


Figure 180.Strain data from Run 3: B3240

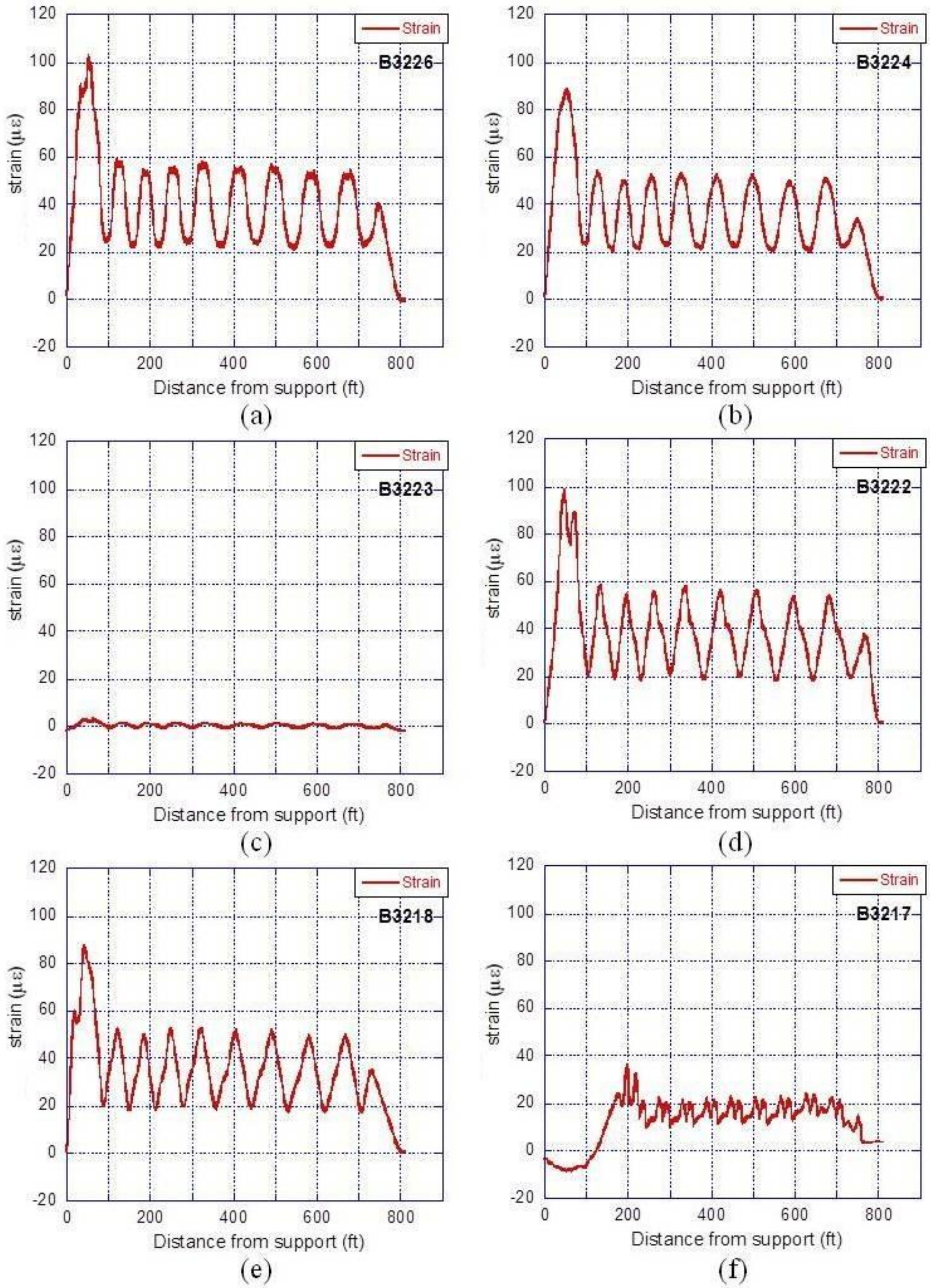


Figure 181.Strain data from Run 4: *(a)* B3226,*(b)* B3224,*(c)* B3223,*(d)* B3222,*(e)* B3218, and*(f)* B3217

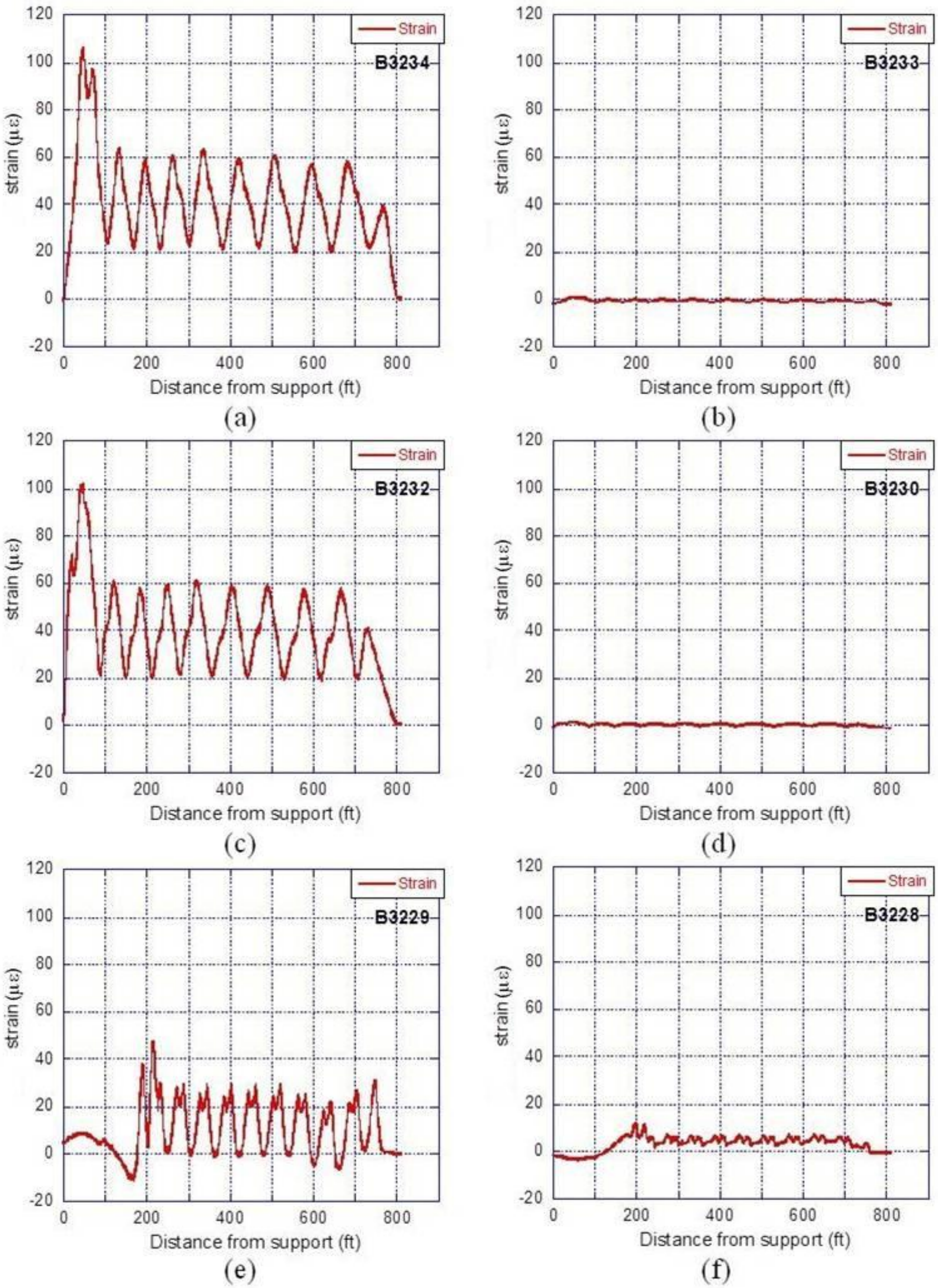


Figure 182.Strain data from Run 4: *(a)* B3234,*(b)* B3233,*(c)* B3232,*(d)* B3230,*(e)* B3229, and*(f)* B3228

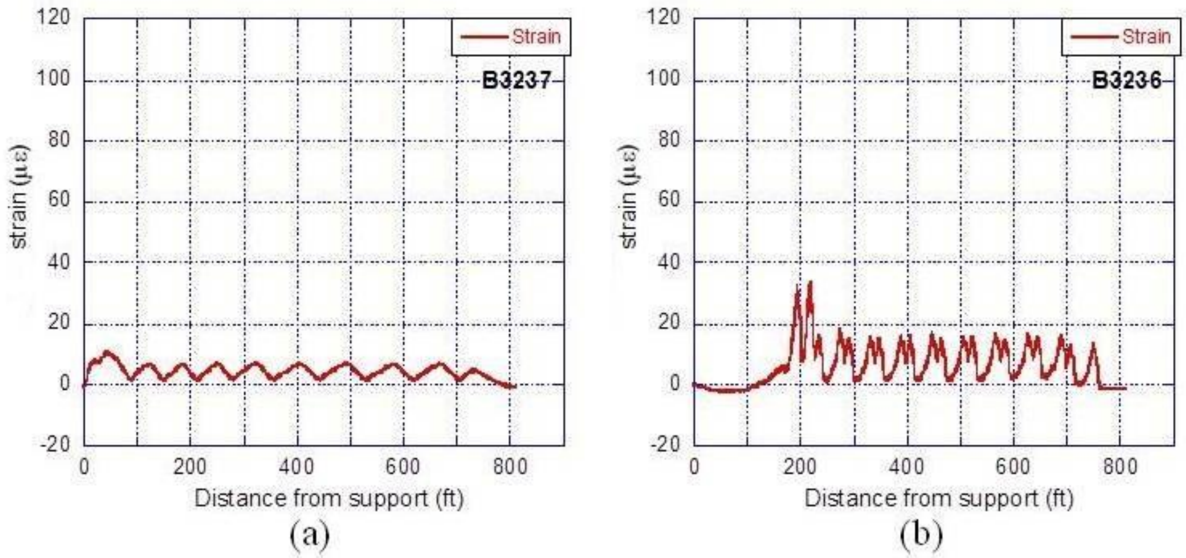


Figure 183.Strain data from Run 4: *(a)* B3237 and*(b)* B3236

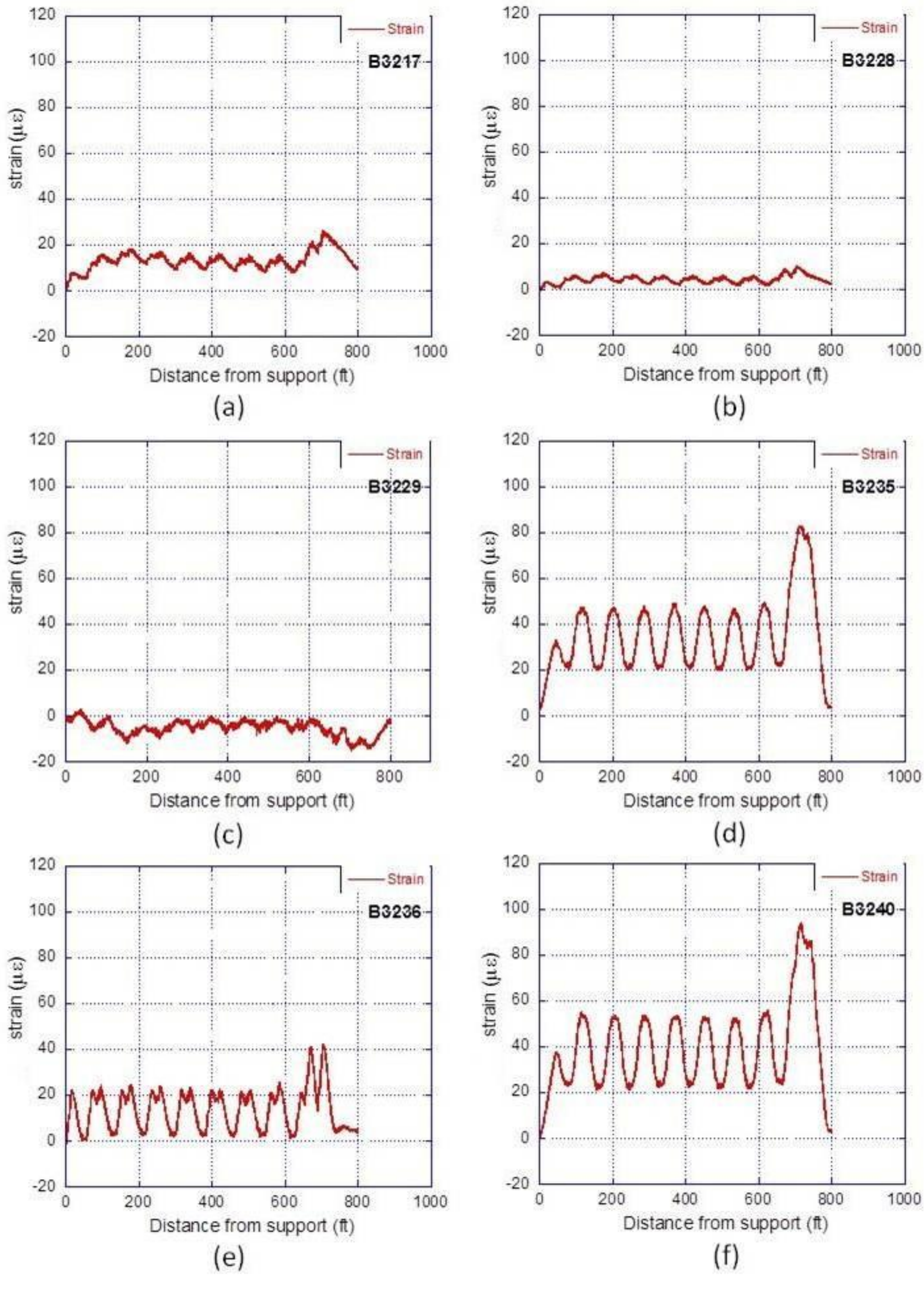


Figure 184.Strain data from Run 5: *(a)* B3217,*(b)* B3228,*(c)* B3229,*(d)* B3235,*(e)* B3236, and *(f)* B3240

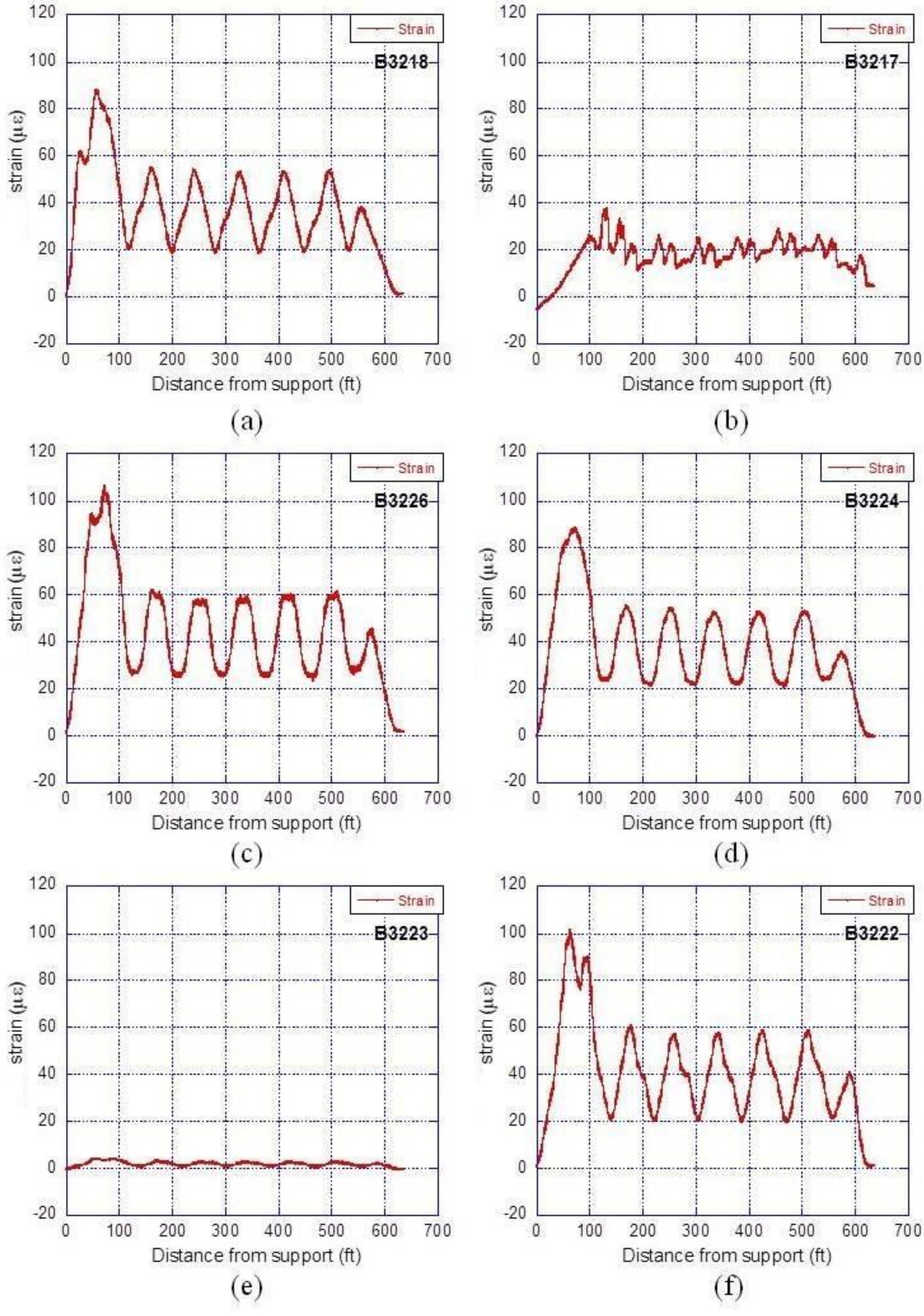


Figure 185.Strain data from Run 6: *(a)* B3218,*(b)* B3217,*(c)* B3226,*(d)* B3224,*(e)* B3223, and*(f)* B3222

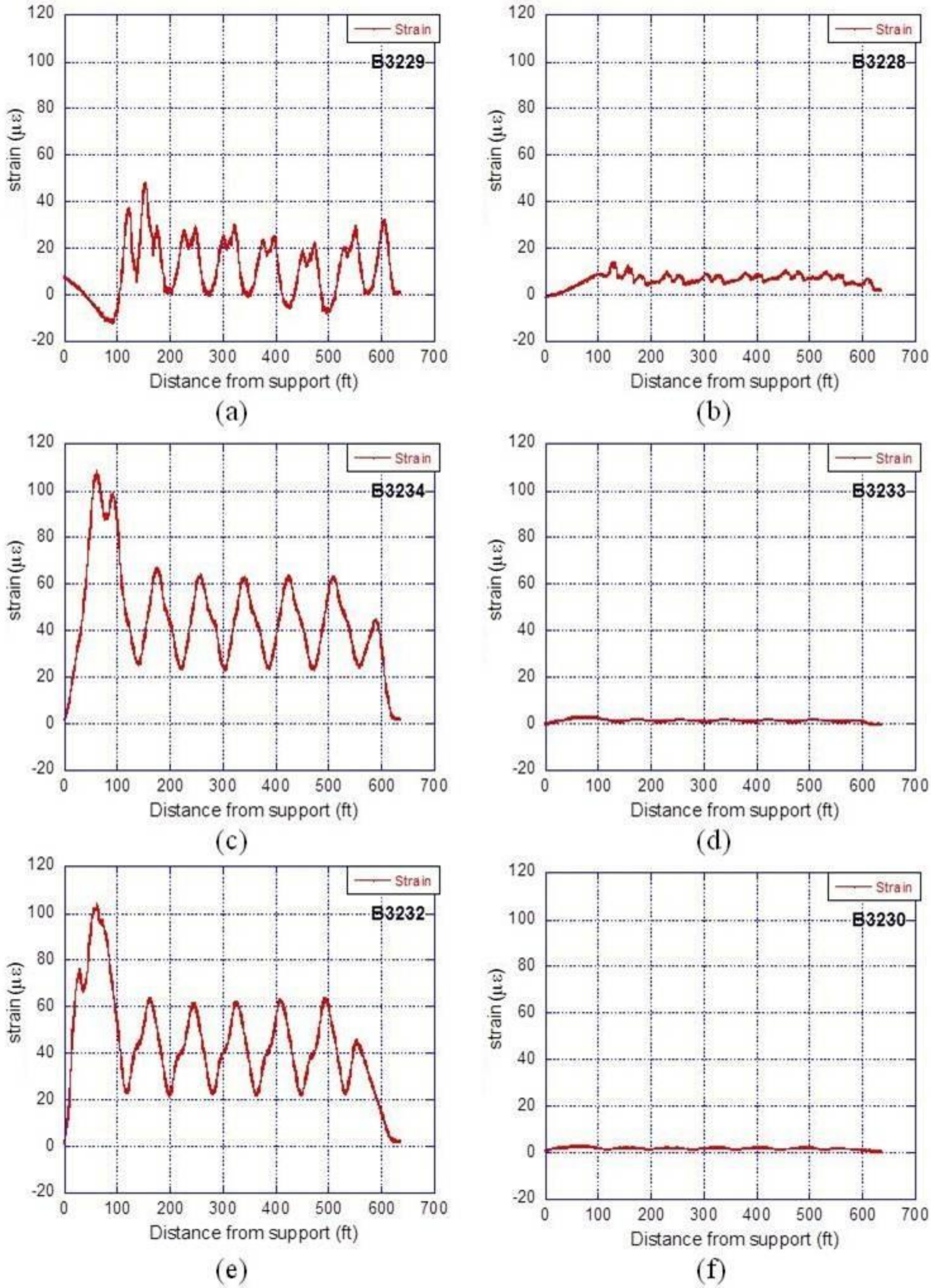


Figure 186.Strain data from Run 6: *(a)* B3229,*(b)* B3228,*(c)* B3234,*(d)* B3233,*(e)* B3232, and*(f)* B3230

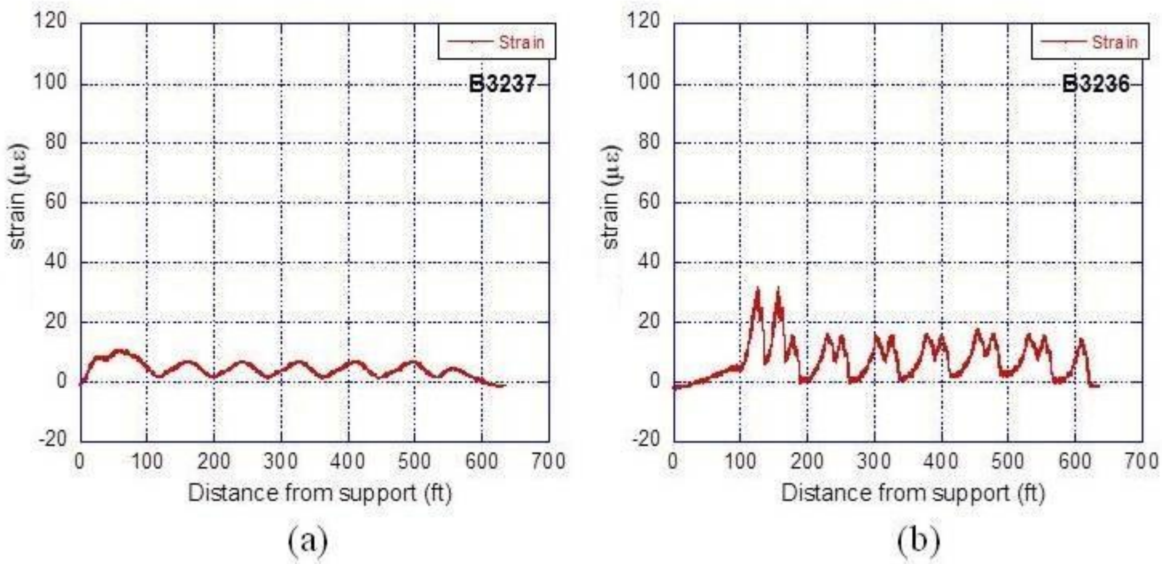


Figure 187.Strain data from Run 6: *(a)* B3237 and*(b)* B3236

APPENDIX F: 286 PROJECT RETROFIT COST ESTIMATE – MAIN LINE MP 15.95

The total cost consists of structural steel repair cost, near white blast cleaning and painting cost, pollution control cost, temporary shielding cost, Barge or man-lift rental cost and flagmen or railroad safety cost.

The structural steel repair cost was calculated by the unit price times the quantity (lbs). The blast cleaning and painting cost was based on the RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004) considering 4% per year inflation. The pollution control cost is same as the blast cleaning and painting cost. The temporary shielding cost included material cost, labor cost and equipment cost. We considered 10% contingencies when calculating the temporary shielding cost. The barge or man-lift included the rental cost and mobilization cost. The rental cost was calculated by the unit cost times the quantity (days). Similarly, the flagmen or railroad safety cost was calculated by the unit cost times the quantity (days).

The following screenshots were taken from the spreadsheet the research team used for the cost calculation.

Cost Analysis Based On Load Rating Using The AREMA Specifications

Cost Estimate for Structural Steel Repairs* to Main Line MP 15.95				
*Addition of Cover Plates w/100% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$200,000	$ 200,000
STRUCTRAL STEEL REPAIR	LBS	600	$20.00	$ 12,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$8,050	$ 8,050
POLLUTION CONTROL	LUMP SUM	LS	$8,050	$ 8,050
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 433,100
	Additonal Cost for Working on Railroad			$ 43,310
			TOTAL COST =	**$ 480,000**
			Add 10% =	**$ 530,000**

Structural Steel Repir Quantities					
Span	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	75.5	1.17	0.0125	1.10	
			TOTAL =	1.10	CF
				600	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,500
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,875
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,800.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,903.75
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,703.75
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,891.90
			2006 Cost =	$5,087.58
			2007 Cost =	$5,291.08
			2008 Cost =	$5,502.72
			2009 Cost =	$5,722.83
			2010 Cost =	$5,951.74
			2011 Cost =	$6,189.81
			2012 Cost =	$6,437.41
4% per year inflation =	$6,437.41			
100% for working over Railroad =	$6,437.41			
25% for Overhead and Profit =	$1,609.35			
TOTAL COST =	**$8,050.00**			
per span				
TOTAL COST =	**$8,050**			

Bridge Width =	18.5	FT			
Bridge Length =	75.5	FT			
Overhang =	3	FT			
Area =	1,849.75	SF			
	205.53	SY			
	51.3819444	SY			
	260	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 14,040	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 190,240	
			10% Contingencies =	$ 9,512.0	
			TOTAL LUMP SUM COST =	**$ 200,000**	

Cost Estimate for Structural Steel Repairs* to Main Line MP 15.95				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$200,000	$ 200,000
STRUCTRAL STEEL REPAIR	LBS	2,100	$20.00	$ 42,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$8,050	$ 8,050
POLLUTION CONTROL	LUMP SUM	LS	$8,050	$ 8,050
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 463,100
	Additonal Cost for Working on Railroad			$ 46,310
			TOTAL COST =	**$ 510,000**
			Add 10% =	**$ 570,000**

Structural Steel Repir Quantities					
Span	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	75.5	1.17	0.042	3.68	
2	17.3	1.17	0.021	0.42	
			TOTAL =	4.10	CF
				2,100	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,500
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,875
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,800.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,903.75
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,703.75
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,891.90
			2006 Cost =	$5,087.58
			2007 Cost =	$5,291.08
			2008 Cost =	$5,502.72
			2009 Cost =	$5,722.83
			2010 Cost =	$5,951.74
			2011 Cost =	$6,189.81
			2012 Cost =	$6,437.41
4% per year inflation =	$6,437.41			
100% for working over Railroad =	$6,437.41			
25% for Overhead and Profit =	$1,609.35			
TOTAL COST =	**$8,050.00**			
per span				
TOTAL COST =	**$8,050**			

Bridge Width =	18.5	FT			
Bridge Length =	75.5	FT			
Overhang =	3	FT			
Area =	1,849.75	SF			
	205.53	SY			
	51.3819444	SY			
	260	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 14,040	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 190,240	
			10% Contingencies =	$ 9,512.0	
			TOTAL LUMP SUM COST =	**$ 200,000**	

APPENDIX G: 286 PROJECT RETROFIT COST ESTIMATE – MAIN LINE MP 15.14

Cost Analysis Based On Load Rating Using The FE Model

Cost Estimate for Structural Steel Repairs* to MP 15.14				
*Addition of Cover Plates- 80% & 100 % D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$397,000	$ 397,000
STRUCTRAL STEEL REPAIR	LBS	5,000	$20.00	$ 100,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$50,750	$ 50,750
POLLUTION CONTROL	LUMP SUM	LS	$50,750	$ 50,750
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 803,500
	Additonal Cost for Working on Railroad			$ 80,350
			TOTAL COST =	**$ 890,000**
			Add 10% =	**$ 980,000**

Structural Steel Repir Quantities for Main Line MP 15.14 Bridge		
Span	Quantity (CF	Repair Type
1	1.38	Repair web holes, replace bottom flange angles
2	3.69	Replace cover plates and stiffeners
3	2.48	Replace north side of G3
4	1.68	Repair floorbeams
5	0.08	Repair weld plates
6	0.62	Repair weld plates
7	0.08	Repair weld plates
TOTAL =	10.01	CF
	5000	**LBS**

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,250
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,588
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,620.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,613.38
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,233.38
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,402.71
			2006 Cost =	$4,578.82
			2007 Cost =	$4,761.97
			2008 Cost =	$4,952.45
			2009 Cost =	$5,150.55
			2010 Cost =	$5,356.57
			2011 Cost =	$5,570.83
			2012 Cost =	$5,793.67
4% per year inflation =	$5,793.67			
100% for working over Railroad =	$5,793.67			
25% for Overhead and Profit =	$1,448.42			
TOTAL COST =	**$7,250.00**			
per span				
TOTAL COST =	**$50,750**			

Bridge Width =	18.5	FT			
Bridge Length =	1095.5	FT			
Overhang =	3	FT			
Area =	26,839.75	SF			
	2,982.19	SY			
	745.548611	SY			
	3,730	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 201,420	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 377,620	
			10% Contingencies =	$ 18,881.0	
			TOTAL LUMP SUM COST =	**$ 397,000**	

Cost Analysis Based On Load Rating Using The AREMA Specifications

Cost Estimate for Structural Steel Repairs* to MP 15.14				
*Addition of Cover Plates- 100% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$397,000	$ 397,000
STRUCTRAL STEEL REPAIR	LBS	34,800	$20.00	$ 696,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$50,750	$ 50,750
POLLUTION CONTROL	LUMP SUM	LS	$50,750	$ 50,750
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 1,399,500
	Additonal Cost for Working on Railroad			$ 139,950
			TOTAL COST =	**$ 1,540,000**
			Add 10% =	**$ 1,700,000**

Structural Steel Repir Quantities for Main Line MP 15.14 Bridge		
Span	Quantity (CF)	Repair Type
1	1.38	Repair web holes, replace bottom flange angles
2	3.69	Replace cover plates and stiffeners
3	2.48	Replace north side of G3
4	1.68	Repair floorbeams
5	0.08	Repair weld plates
6	0.62	Repair weld plates
7	0.08	Repair weld plates
8	60.86	add 0.5" steel cover plate to the bottom flange of G30 to G39 (hole length)
TOTAL =	70.87	CF
	34800	**LBS**

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,250
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,588
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,620.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,613.38
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,233.38
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,402.71
			2006 Cost =	$4,578.82
			2007 Cost =	$4,761.97
			2008 Cost =	$4,952.45
			2009 Cost =	$5,150.55
			2010 Cost =	$5,356.57
			2011 Cost =	$5,570.83
			2012 Cost =	$5,793.67
4% per year inflation =	$5,793.67			
100% for working over Railroad =	$5,793.67			
25% for Overhead and Profit =	$1,448.42			
TOTAL COST =	**$7,250.00**			
per span				
TOTAL COST =	**$50,750**			

Bridge Width =	18.5	FT			
Bridge Length =	1095.5	FT			
Overhang =	3	FT			
Area =	26,839.75	SF			
	2,982.19	SY			
	745.548611	SY			
	3,730	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 201,420	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 377,620	
			10% Contingencies =	$18,881.0	
			TOTAL LUMP SUM C(	**$ 397,000**	

Cost Estimate for Structural Steel Repairs* to MP 15.14				
*Addition of Cover Plates- 80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$397,000	$ 397,000
STRUCTRAL STEEL REPAIR	LBS	118,300	$20.00	$ 2,366,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$50,750	$ 50,750
POLLUTION CONTROL	LUMP SUM	LS	$50,750	$ 50,750
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 3,069,500
	Additonal Cost for Working on Railroad			$ 306,950
			TOTAL COST =	**$ 3,380,000**
			Add 10% =	**$ 3,720,000**

Structural Steel Repir Quantities for Main Line MP 15.14 Bridge		
Span	Quantity (CF)	Repair Type
1	1.38	Repair web holes, replace bottom flange angles
2	3.69	Replace cover plates and stiffeners
3	2.48	Replace north side of G3
4	1.68	Repair floorbeams
5	0.08	Repair weld plates
6	0.62	Repair weld plates
7	0.08	Repair weld plates
8	48.69	add 0.4" steel cover plates to the bottom flange of center girder under active track
9	182.58	add 1.5" steel cover plates to the bottom flange of G30 to G39
TOTAL =	241.28	CF
	118300	**LBS**

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,250
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,588
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,620.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,613.38
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,233.38
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,402.71
			2006 Cost =	$4,578.82
			2007 Cost =	$4,761.97
			2008 Cost =	$4,952.45
			2009 Cost =	$5,150.55
			2010 Cost =	$5,356.57
			2011 Cost =	$5,570.83
			2012 Cost =	$5,793.67
4% per year inflation =	$5,793.67			
100% for working over Railroad =	$5,793.67			
25% for Overhead and Profit =	$1,448.42			
TOTAL COST =	**$7,250.00**			
per span				
TOTAL COST =	**$50,750**			

Bridge Width =	18.5	FT			
Bridge Length =	1095.5	FT			
Overhang =	3	FT			
Area =	26,839.75	SF			
	2,982.19	SY			
	745.548611	SY			
	3,730	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 201,420	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 377,620	
			10% Contingencies =	$18,881.0	
			TOTAL LUMP SUM COST =	**$ 397,000**	

APPENDIX H: 286 PROJECT RETROFIT COST ESTIMATE – BERGEN COUNTY LINE MP 5.48 (HX DRAW)

Cost Analysis Based On Load Rating Using The FE Model And AREMA Specifications

Cost Estimate for Structural Steel Repairs* to HX Drawbridge				
*Addition of Cover Plates - 100% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$397,000	$ 397,000
STRUCTRAL STEEL REPAIR	LBS	73,500	$20.00	$ 1,470,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$164,220	$ 164,220
POLLUTION CONTROL	LUMP SUM	LS	$164,220	$ 164,220
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 2,400,440
Additonal Cost for Working over Water and on Railroad				$ 240,044
			TOTAL COST =	**$ 2,650,000**
			Add 10% =	**$ 2,920,000**

Structural Steel Repir Quantities for HX Drawbridge

Span	Track	Length (FT)	Width (FT)	Thickness (FT	Quantity (CF	TOTAL (CF)
1	1	40.50	1.25	0.09	9.49	
	2	40.50	1.25	0.01	0.53	
					TOTAL =	**10.02**
2	2	10.00	1.25	0.10	4.95	
	2	5.60	1.25	0.10	2.92	
					TOTAL =	**8.00**
3	1	20.00	1.25	0.02	0.52	
	1	20.00	1.25	0.07	1.82	
	2	20.00	1.25	0.02	0.52	
	2	20.00	1.25	0.03	0.78	
					TOTAL =	**3.65**
4	1	60.00	1.25	0.06	9.38	
	2	60.00	1.25	0.01	1.56	
					TOTAL =	**10.94**
5	1	40.50	1.25	0.05	5.27	
	2	40.50	1.25	0.01	1.05	
	2	0.50	1.25	0.03	0.02	
					TOTAL =	**6.35**
6	1	40.50	1.25	0.05	5.27	
	2	40.50	1.25	0.06	3.16	
					TOTAL =	**8.44**
7	1	40.50	1.25	0.05	5.27	
	2	40.50	1.25	0.04	4.22	
					TOTAL =	**9.49**
8	1	40.50	1.25	0.06	6.33	
	2	40.50	1.25	0.06	3.16	
					TOTAL =	**9.49**
9	FB	16.00	1.17	0.05	0.97	
		8.00	1.17	0.04	0.39	
		2.00	1.17	0.02	0.05	
	Girders	7.00	1.50	0.05	1.09	
		53.00	1.50	0.04	6.63	
					TOTAL =	**9.13**

10	1	40.50	1.25	0.06	6.33		
	2	40.50	1.25	0.06	6.33		
					TOTAL =	**12.66**	
11	1	13.00	1.25	0.02	1.35		
		28.00	1.25	0.04	5.83		
					TOTAL =	**7.19**	
12	1	45.00	1.17	0.02	4.38		
					TOTAL =	**4.38**	
13	1	24.17	1.17	0.02	4.70		
					TOTAL =	**4.70**	
14	Girder	23.00	1.25	0.04	4.79		
		18.00	1.25	0.02	1.88		
					TOTAL =	**6.67**	
15	1	40.50	1.25	0.06	6.33		
	2	0.50	1.25	0.10	0.07		
	2	0.50	1.25	0.06	0.04		
					TOTAL =	**6.43**	
16	1	11.00	1.25	0.13	1.72		
		5.67	1.25	0.10	1.40		
		10.00	1.25	0.10	2.60		
	2	9.58	1.25	0.05	0.62		
		5.67	1.25	0.05	0.37		
		5.67	1.25	0.10	2.10		
		10.00	1.25	0.10	3.91		
					TOTAL =	**12.73**	
17	1	5.67	1.25	0.10	2.80		
	1	10.00	1.25	0.10	5.21		
	2	5.67	1.25	0.10	2.80		
	2	10.00	1.25	0.10	5.21		
					TOTAL =	**16.02**	
					QUANTITY TOTAL =	**146.27**	**CF**
					ROUNDED =	**150**	**CF**
					Pounds of Steel =	**73,500**	**lbs**

Near White Blast Cleaning and Painting Quantity

Span	Track	Length (FT)	Width (FT)	Quantity (SF)	TOTAL (SF)
1	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
2	2	10	1.25	50.00	
	2	5.6	1.25	28.00	
				TOTAL =	**78.00**
3	1	20	1.25	25.00	
	1	20	1.25	25.00	
	2	20	1.25	25.00	
	2	20	1.25	25.00	
				TOTAL =	**100.00**
4	1	60	1.25	150.00	
	2	60	1.25	150.00	
				TOTAL =	**300.00**
5	1	40.5	1.25	101.25	
	2	40.5	1.25	101.25	
	2	0.5	1.25	0.63	
				TOTAL =	**203.13**
6	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
7	1	40.5	1.25	101.25	
	2	40.5	1.25	101.25	
				TOTAL =	**202.50**
8	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
9	FB	16	1.17	18.67	
		8	1.17	9.33	
		2	1.17	2.33	
	Girders	7	1.5	21.00	
		53	1.5	159.00	
				TOTAL =	**210.33**

10	1	40.5	1.25	101.25		
	2	40.5	1.25	101.25		
				TOTAL =	**202.50**	
11	1	13	1.25	65.00		
		28	1.25	140.00		
				TOTAL =	**205.00**	
12	1	45	1.17	210.00		
				TOTAL =	**210.00**	
13	1	24.167	1.17	225.56		
				TOTAL =	**225.56**	
14	Girder	23	1.25	115.00		
		18	1.25	90.00		
				TOTAL =	**205.00**	
15	1	40.5	1.25	101.25		
	2	0.5	1.25	0.63		
	2	0.5	1.25	0.63		
				TOTAL =	**102.50**	
16	1	11	1.25	13.75		
		5.67	1.25	14.17		
		10	1.25	25.00		
	2	9.58	1.25	11.98		
		5.67	1.25	7.08		
		5.67	1.25	21.25		
		10	1.25	37.50		
				TOTAL =	**130.73**	
17	1	5.67	1.25	28.33		
	1	10	1.25	50.00		
	2	5.67	1.25	28.33		
	2	10	1.25	50.00		
				TOTAL =	**156.67**	
				QUANTITY TOTAL =	**2987.54**	**SF**
				ROUNDED =	**3,000**	**SF**

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	3,000
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	3,450
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$2,160.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$3,484.50
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$5,644.50
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$5,870.28
			2006 Cost =	$6,105.09
			2007 Cost =	$6,349.29
			2008 Cost =	$6,603.27
			2009 Cost =	$6,867.40
			2010 Cost =	$7,142.09
			2011 Cost =	$7,427.78
			2012 Cost =	$7,724.89
4% per year inflation =	$7,724.89			
100% for working over Railroad =	$7,724.89			
25% for Overhead and Profit =	$1,931.22			
TOTAL COST =	**$9,660.00**			
per span				
TOTAL COST =	**$164,220**			

Bridge Width =	18.5	FT			
Bridge Length =	1095.5	FT			
Overhang =	3	FT			
Area =	26,839.75	SF			
	2,982.19	SY			
	745.548611	SY			
	3,730	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 201,420	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 377,620	
			10% Contingencies =	$18,881.0	
			TOTAL LUMP SUM COST =	**$ 397,000**	

Cost Estimate for Structural Steel Repairs* to HX Drawbridge				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$397,000	$ 397,000
STRUCTRAL STEEL REPAIR	LBS	196,000	$20.00	$ 3,920,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$164,220	$ 164,220
POLLUTION CONTROL	LUMP SUM	LS	$164,220	$ 164,220
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 4,850,440
Additonal Cost for Working over Water and on Railroad				$ 485,044
			TOTAL COST =	**$ 5,340,000**
			Add 10% =	**$ 5,880,000**

Structural Steel Repir Quantities for HX Drawbridge

Span	Track	Length (FT)	Width (FT)	Thickness (FT	Quantity (CF	TOTAL (CF)
1	1	40.5	1.25	0.18	17.93	
	2	40.5	1.25	0.09	4.75	
					TOTAL =	**22.68**
2	2	10	1.25	0.18	9.00	
	2	5.6	1.25	0.19	5.32	
					TOTAL =	**15.00**
3	1	20	1.25	0.1	2.6	
	1	20	1.25	0.16	3.91	
	2	20	1.25	0.1	2.6	
	2	20	1.25	0.11	2.86	
					TOTAL =	**11.97**
4	1	60	1.25	0.15	22.50	
	2	60	1.25	0.09	13.50	
					TOTAL =	**36.00**
5	1	40.5	1.25	0.14	14.18	
	2	40.5	1.25	0.09	9.11	
	2	0.5	1.25	0.11	0.07	
					TOTAL =	**23.36**
6	1	40.5	1.25	0.14	14.18	
	2	40.5	1.25	0.15	7.59	
					TOTAL =	**21.77**
7	1	40.5	1.25	0.14	14.18	
	2	40.5	1.25	0.13	13.16	
					TOTAL =	**27.34**
8	1	40.5	1.25	0.15	15.19	
	2	40.5	1.25	0.15	7.59	
					TOTAL =	**22.78**
9	FB	16	1.17	0.14	2.62	
		8	1.17	0.13	1.22	
		2	1.17	0.1	0.23	
	Girders	7	1.5	0.14	2.94	
		53	1.5	0.13	20.67	
					TOTAL =	**27.68**

10	1	40.5	1.25	0.15	15.19		
	2	40.5	1.25	0.15	15.19		
					TOTAL =	**30.38**	
11	1	13	1.25	0.1	6.50		
		28	1.25	0.13	18.20		
					TOTAL =	**24.70**	
12	1	45	1.17	0.1	21.06		
					TOTAL =	**21.06**	
13	1	24.167	1.17	0.1	22.62		
					TOTAL =	**22.62**	
14	Girder	23	1.25	0.13	14.95		
		18	1.25	0.1	9.00		
					TOTAL =	**23.95**	
15	1	40.5	1.25	0.15	15.19		
	2	0.5	1.25	0.19	0.12		
	2	0.5	1.25	0.15	0.09		
					TOTAL =	**15.40**	
16	1	11	1.25	0.21	2.89		
		5.67	1.25	0.18	2.55		
		10	1.25	0.19	4.75		
	2	9.58	1.25	0.14	1.68		
		5.67	1.25	0.14	0.99		
		5.67	1.25	0.18	3.83		
		10	1.25	0.19	7.13		
					TOTAL =	**23.81**	
17	1	5.67	1.25	0.18	5.10		
	1	10	1.25	0.19	9.50		
	2	5.67	1.25	0.18	5.10		
	2	10	1.25	0.19	9.50		
					TOTAL =	**29.21**	
					QUANTITY TOTAL =	**399.69**	**CF**
					ROUNDED =	**400**	**CF**
					Pounds of Steel =	**196,000**	**lbs**

Near White Blast Cleaning and Painting Quantity					
Span	**Track**	**Length (FT)**	**Width (FT)**	**Quantity (SF)**	**TOTAL (SF)**
1	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
2	2	10	1.25	50.00	
	2	5.6	1.25	28.00	
				TOTAL =	**78.00**
3	1	20	1.25	25.00	
	1	20	1.25	25.00	
	2	20	1.25	25.00	
	2	20	1.25	25.00	
				TOTAL =	**100.00**
4	1	60	1.25	150.00	
	2	60	1.25	150.00	
				TOTAL =	**300.00**
5	1	40.5	1.25	101.25	
	2	40.5	1.25	101.25	
	2	0.5	1.25	0.63	
				TOTAL =	**203.13**
6	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
7	1	40.5	1.25	101.25	
	2	40.5	1.25	101.25	
				TOTAL =	**202.50**
8	1	40.5	1.25	101.25	
	2	40.5	1.25	50.63	
				TOTAL =	**151.88**
9	FB	16	1.17	18.67	
		8	1.17	9.33	
		2	1.17	2.33	
	Girders	7	1.5	21.00	
		53	1.5	159.00	
				TOTAL =	**210.33**

10	1	40.5	1.25	101.25		
	2	40.5	1.25	101.25		
				TOTAL =	**202.50**	
11	1	13	1.25	65.00		
		28	1.25	140.00		
				TOTAL =	**205.00**	
12	1	45	1.17	210.00		
				TOTAL =	**210.00**	
13	1	24.17	1.17	225.56		
				TOTAL =	**225.56**	
14	Girder	23	1.25	115.00		
		18	1.25	90.00		
				TOTAL =	**205.00**	
15	1	40.5	1.25	101.25		
	2	0.5	1.25	0.63		
	2	0.5	1.25	0.63		
				TOTAL =	**102.50**	
16	1	11	1.25	13.75		
		5.67	1.25	14.17		
		10	1.25	25.00		
	2	9.58	1.25	11.98		
		5.67	1.25	7.08		
		5.67	1.25	21.25		
		10	1.25	37.50		
				TOTAL =	**130.73**	
17	1	5.67	1.25	28.33		
	1	10	1.25	50.00		
	2	5.67	1.25	28.33		
	2	10	1.25	50.00		
				TOTAL =	**156.67**	
				QUANTITY TOTAL =	**2987.54**	**SF**
				ROUNDED =	**3,000**	**SF**

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	3,000
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	3,450
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$2,160.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$3,484.50
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$5,644.50
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$5,870.28
			2006 Cost =	$6,105.09
			2007 Cost =	$6,349.29
			2008 Cost =	$6,603.27
			2009 Cost =	$6,867.40
			2010 Cost =	$7,142.09
			2011 Cost =	$7,427.78
			2012 Cost =	$7,724.89
4% per year inflation =	$7,724.89			
100% for working over Railroad =	$7,724.89			
25% for Overhead and Profit =	$1,931.22			
TOTAL COST =	**$9,660.00**			
per span				
TOTAL COST =	**$164,220**			

Bridge Width =	18.5	FT			
Bridge Length =	1095.5	FT			
Overhang =	3	FT			
Area =	26,839.75	SF			
	2,982.19	SY			
	745.548611	SY			
	3,730	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 201,420	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 377,620	
			10% Contingencies =	$ 18,881.0	
			TOTAL LUMP SUM COST =	**$ 397,000**	

APPENDIX I: 286 PROJECT RETROFIT COST ESTIMATE – RARITAN VALLEY LINE MP 31.15

Cost Analysis Based On Load Rating Using The FE Model

Cost Estimate for Structural Steel Repairs* to Raritan Valley MP 31.15				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$217,000	$ 217,000
STRUCTRAL STEEL REPAIR	LBS	11,100	$20.00	$ 222,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$32,200	$ 32,200
POLLUTION CONTROL	LUMP SUM	LS	$32,200	$ 32,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 708,400
	Additonal Cost for Working on Railroad			$ 70,840
			TOTAL COST =	**$ 780,000**
			Add 10% =	**$ 860,000**

Structural Steel Repir Quantities					
Span	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	40	1.17	0.03	5.62	
2	40	1.17	0.03	5.62	
3	40	1.17	0.03	5.62	
4	40	1.17	0.03	5.62	
			TOTAL =	22.46	CF
				11,100	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,500
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,875
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,800.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,903.75
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,703.75
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,891.90
			2006 Cost =	$5,087.58
			2007 Cost =	$5,291.08
			2008 Cost =	$5,502.72
			2009 Cost =	$5,722.83
			2010 Cost =	$5,951.74
			2011 Cost =	$6,189.81
			2012 Cost =	$6,437.41
4% per year inflation =	$6,437.41			
100% for working over Railroad =	$6,437.41			
25% for Overhead and Profit =	$1,609.35			
TOTAL COST =	**$8,050.00**			
per span				
TOTAL COST =	**$32,200**			

Bridge Width =	18.5	FT			
Bridge Length =	160	FT			
Overhang =	3	FT			
Area =	3,920.00	SF			
	435.56	SY			
	108.888889	SY			
	550	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 29,700	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 205,900	
			10% Contingencies =	$ 10,295.0	
			TOTAL LUMP SUM COST =	**$ 217,000**	

Cost Analysis Based On Load Rating Using The AREMA Specifications

Cost Estimate for Structural Steel Repairs* to Raritan Valley MP 31.15				
*Addition of Cover Plates w/100% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$217,000	$ 217,000
STRUCTRAL STEEL REPAIR	LBS	7,400	$20.00	$ 148,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$32,200	$ 32,200
POLLUTION CONTROL	LUMP SUM	LS	$32,200	$ 32,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 634,400
	Additonal Cost for Working on Railroad			$ 63,440
			TOTAL COST =	**$ 700,000**
			Add 10% =	**$ 770,000**

Structural Steel Repir Quantities					
Span	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	40	1.17	0.02	3.74	
2	40	1.17	0.02	3.74	
3	40	1.17	0.02	3.74	
4	40	1.17	0.02	3.74	
			TOTAL =	14.98	CF
				7,400	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,500
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,875
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,800.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,903.75
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,703.75
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,891.90
			2006 Cost =	$5,087.58
			2007 Cost =	$5,291.08
			2008 Cost =	$5,502.72
			2009 Cost =	$5,722.83
			2010 Cost =	$5,951.74
			2011 Cost =	$6,189.81
			2012 Cost =	$6,437.41
4% per year inflation =	$6,437.41			
100% for working over Railroad =	$6,437.41			
25% for Overhead and Profit =	$1,609.35			
TOTAL COST =	**$8,050.00**			
per span				
TOTAL COST =	**$32,200**			

Bridge Width =	18.5	FT			
Bridge Length =	160	FT			
Overhang =	3	FT			
Area =	3,920.00	SF			
	435.56	SY			
	108.888889	SY			
	550	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 29,700	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 205,900	
			10% Contingencies =	$10,295.0	
			TOTAL LUMP SUM COST =	**$ 217,000**	

Cost Estimate for Structural Steel Repairs* to Raritan Valley MP 31.15				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$217,000	$ 217,000
STRUCTRAL STEEL REPAIR	LBS	16,900	$20.00	$ 338,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$32,200	$ 32,200
POLLUTION CONTROL	LUMP SUM	LS	$32,200	$ 32,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 824,400
	Additonal Cost for Working on Railroad			$ 82,440
			TOTAL COST =	**$ 910,000**
			Add 10% =	**$ 1,010,000**

Structural Steel Repir Quantities					
Span	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	40	1.17	0.046	8.58	
2	40	1.17	0.046	8.58	
3	40	1.17	0.046	8.58	
4	40	1.17	0.046	8.58	
			TOTAL =	34.32	CF
				16,900	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	2,500
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	2,875
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$1,800.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$2,903.75
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$4,703.75
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$4,891.90
			2006 Cost =	$5,087.58
			2007 Cost =	$5,291.08
			2008 Cost =	$5,502.72
			2009 Cost =	$5,722.83
			2010 Cost =	$5,951.74
			2011 Cost =	$6,189.81
			2012 Cost =	$6,437.41
4% per year inflation =	$6,437.41			
100% for working over Railroad =	$6,437.41			
25% for Overhead and Profit =	$1,609.35			
TOTAL COST =	**$8,050.00**			
per span				
TOTAL COST =	**$32,200**			

Bridge Width =	18.5	FT			
Bridge Length =	160	FT			
Overhang =	3	FT			
Area =	3,920.00	SF			
	435.56	SY			
	108.888889	SY			
	550	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 29,700	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 205,900	
			10% Contingencies =	$10,295.0	
			TOTAL LUMP SUM COST =	**$ 217,000**	

APPENDIX J: 286 PROJECT RETROFIT COST ESTIMATE – NORTH JERSEY COAST LINE MP 0.39

Cost Analysis Based On Load Rating Using The FE Model

Cost Estimate for Structural Steel Repairs* to North Jersey Coast Line MP 0.39				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$441,000	$ 441,000
STRUCTRAL STEEL REPAIR	LBS	345,700	$20.00	$ 6,914,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$193,200	$ 193,200
POLLUTION CONTROL	LUMP SUM	LS	$193,200	$ 193,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 7,946,400
	Additonal Cost for Working on Railroad			$ 794,640
			TOTAL COST =	**$ 8,750,000**
			Add 10% =	**$ 9,630,000**

Structural Steel Repir Quantities					
Plate	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	88	1.67	0.08	705.41	
2	39	1.67	0.08	312.62	
			TOTAL =	705.41	CF
				345,700	LBS

Bridge Width =	18.5	FT			
Bridge Length =	1320	FT			
Overhang =	3	FT			
Area =	32,340.00	SF			
	3,593.33	SY			
	898.333333	SY			
	4,500	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 243,000	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 419,200	
			10% Contingencies =	$ 20,960.0	
			TOTAL LUMP SUM COST =	**$ 441,000**	

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	4,000
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	4,600
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$2,880.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$4,646.00
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$7,526.00
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$7,827.04
			2006 Cost =	$8,140.12
			2007 Cost =	$8,465.73
			2008 Cost =	$8,804.36
			2009 Cost =	$9,156.53
			2010 Cost =	$9,522.79
			2011 Cost =	$9,903.70
			2012 Cost =	$10,299.85
4% per year inflation =	$10,299.85			
100% for working over Railroad =	$10,299.85			
25% for Overhead and Profit =	$2,574.96			
TOTAL COST =	**$12,880.00**			
per span				
TOTAL COST =	**$193,200**			

Cost Analysis Based On Load Rating Using The AREMA Specifications

Cost Estimate for Structural Steel Repairs* to North Jersey Coast Line MP 0.39				
*Addition of Cover Plates w/100% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$441,000	$ 441,000
STRUCTRAL STEEL REPAIR	LBS	120,300	$20.00	$ 2,406,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$193,200	$ 193,200
POLLUTION CONTROL	LUMP SUM	LS	$193,200	$ 193,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 3,438,400
	Additonal Cost for Working on Railroad			$ 343,840
			TOTAL COST =	**$ 3,790,000**
			Add 10% =	**$ 4,170,000**

Structural Steel Repir Quantities					
Plate	**Length (FT)**	**Width (FT)**	**Thickness (FT)**	**Quantity (CF)**	
1	88	1.67	0.08	705.41	
2	49	1.67	0.05	245.49	
			TOTAL =	245.49	CF
				120,300	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	4,000
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	4,600
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$2,880.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$4,646.00
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$7,526.00
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$7,827.04
			2006 Cost =	$8,140.12
			2007 Cost =	$8,465.73
			2008 Cost =	$8,804.36
			2009 Cost =	$9,156.53
			2010 Cost =	$9,522.79
			2011 Cost =	$9,903.70
			2012 Cost =	$10,299.85
4% per year inflation =	$10,299.85			
100% for working over Railroad =	$10,299.85			
25% for Overhead and Profit =	$2,574.96			
TOTAL COST =	**$12,880.00**			
per span				
TOTAL COST =	**$193,200**			

Bridge Width =	18.5	FT			
Bridge Length =	1320	FT			
Overhang =	3	FT			
Area =	32,340.00	SF			
	3,593.33	SY			
	898.333333	SY			
	4,500	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 243,000	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 419,200	
			10% Contingencies =	$20,960.0	
			TOTAL LUMP SUM COST =	**$ 441,000**	

Cost Estimate for Structural Steel Repairs* to North Jersey Coast Line MP 0.39				
*Addition of Cover Plates w/80% D/C Ratio				
	Unit	Quantity	Unit Cost	Quantity Cost
TEMPORARY SHIELDING	LUMP SUM	LS	$441,000	$ 441,000
STRUCTRAL STEEL REPAIR	LBS	466,000	$20.00	$ 9,320,000
NEAR WHITE BLAST CLEANING/PAINTING	LUMP SUM	LS	$193,200	$ 193,200
POLLUTION CONTROL	LUMP SUM	LS	$193,200	$ 193,200
Barge/Man-Lift Rental	per day	45	$3,000	$135,000
Barge/Man-Lift Rental (Mobilization)	LS	LS	$25,000	$25,000
Flagmen/Railroad Safety Costs	per day	45	$1,000	$45,000
			SUBTOTAL	$ 10,352,400
	Additonal Cost for Working on Railroad			$ 1,035,240
			TOTAL COST =	**$ 11,390,000**
			Add 10% =	**$ 12,530,000**

Structural Steel Repir Quantities

Plate	Length (FT)	Width (FT)	Thickness (FT)	Quantity (CF)	
1	88	1.67	0.08	705.41	
2	49	1.67	0.05	245.49	
			TOTAL =	950.90	CF
				466,000	LBS

Blast Cleaning Cost Estimate				
Cost Backup: (RS Means Site Work & Landscape Cost Data, 23rd Annual Edition (2004)				
Average Trenton City Cost Index =	110		Total S.F. to Clean =	4,000
Average Daily Output (S.F) =	1200		Plus 15% for Misc =	4,600
Average Labor Hours per S.F. =	0.027		2004 Total Materials Cost =	$2,880.00
Blast Cost per hour for E-11 Crew =	$151.36		2004 Labor Cost =	$4,646.00
2004 Bare Materials Cost per S.F. =	$0.72		Total 2004 Cost =	$7,526.00
2004 Bare Labor Costs per S.F. =	$1.01		2005 Cost =	$7,827.04
			2006 Cost =	$8,140.12
			2007 Cost =	$8,465.73
			2008 Cost =	$8,804.36
			2009 Cost =	$9,156.53
			2010 Cost =	$9,522.79
			2011 Cost =	$9,903.70
			2012 Cost =	$10,299.85
4% per year inflation =	$10,299.85			
100% for working over Railroad =	$10,299.85			
25% for Overhead and Profit =	$2,574.96			
TOTAL COST =	**$12,880.00**			
per span				
TOTAL COST =	**$193,200**			

Bridge Width =	18.5	FT			
Bridge Length =	1320	FT			
Overhang =	3	FT			
Area =	32,340.00	SF			
	3,593.33	SY			
	898.333333	SY			
	4,500	SY			
Backup for Estimated Unit Price					
Material Cost:					
	Assume $6.00/Board-FT			$ 54.00	SY
	Cost of Material =			$ 243,000	
Labor Cost :					
	6 Man Crew for 30 days (240 HRS)			$ 50,400	
	Add 200% for Working over Water/Rail =			$ 100,800	
				$ 151,200	
Equipment Cost:					
	Barge for 30 days =			$ 25,000	
			Subtotal =	$ 419,200	
			10% Contingencies =	$ 20,960.0	
			TOTAL LUMP SUM COST =	**$ 441,000**	