

# Lessons Learned from Diagnostic Load Testing of 100-Year-Old Elevated Mass Transit Structures

R. W. KRITZLER, S. G. PINJARKAR, R. A. ROLSING, AND P. O. MCCARTHY

A comprehensive study has been undertaken by the Chicago Transit Authority and its consultants for condition assessment of the rapid-transit system's elevated steel structures. This study included a series of full-scale diagnostic load tests performed to evaluate actual behavior of these structures under train loading. Static and dynamic load tests, using an empty four-car test train, were performed on four different line segments, representing different structural systems. Dynamic monitoring of tensile bending stresses was also conducted for normal rush-hour traffic, providing actual variable stress and average response data to be used for fatigue life assessment. Strain gauge response data for the test structures showed that stringer midspan bending stresses were 36 to 93 percent of nominal calculated stresses, assuming simply supported spans. The measured negative moments at the supports, caused by partial end continuity, were 25 to 86 percent of the fully continuous end moments for three of the test structures. The continuity at the stringer supports is provided by typical riveted full-depth web-shear framing angles. Dynamic test results showed the average measured impact loading for each stringer span to vary from 7 to 34 percent of the respective maximum static test response stresses, compared with American Railway Engineering Association-required design effects of 53 to 59 percent. The measured impact varied inversely with the span length. Train braking test results indicated that longitudinal braking forces are nonuniformly distributed along a length of the track structure exceeding the train length, and the rails are effective in distributing those braking forces across longitudinal structural expansion joints.

Previous load tests of older and apparently deteriorated railroad and highway structures (1) indicate that these structures can carry substantially higher loads than are computed by analytical design methods. The factors that contribute to this increase in strength include unintended continuity, unintended composite action, better lateral load distribution, much smaller impact than is required by design codes, and smaller magnitude of live load stresses used in fatigue rating calculations. Results of a pilot load testing program of a Chicago Transit Authority (CTA) mass transit structure (2) indicated that diagnostic load tests can be successfully conducted for rating and evaluation of older mass transit structures.

The Chicago Rapid Transit System includes approximately 40 mi of elevated steel structures and bridges. As part of a comprehensive Engineering Condition Assessment of the en-

R. W. Kritzler, Raths, Raths & Johnson, Inc., 835 Midway Drive, Willowbrook, Ill. 60521. S. G. Pinjarkar, Desman Associates, 300 West Washington, Suite 1010, Chicago, Ill. 60606. R. A. Rolsing, McDonough Associates, Inc., 224 South Michigan, Chicago, Ill. 60604. P. O. McCarthy, H. W. Lochner, Inc., 20 North Wacker Drive, Suite 1200, Chicago, Ill. 60606.

tire system, a series of full-scale diagnostic load tests were performed to evaluate actual behavior of these structures under train loading. Load tests were performed on four different line segments of the elevated system built from 1891 through 1905.

The test locations were selected to represent variations in structural system and details between each line segment. A test program was specifically designed for each location to evaluate end connection continuity, load distribution, critical stresses, impact loading, fatigue, and longitudinal forces caused by train braking. Both static and dynamic load tests were performed to determine the response characteristics of the track structures. The dynamic tests included both empty four-car test trains and actual rush-hour commuter trains. The results of this diagnostic load testing are currently being used to supplement the engineering analyses and rating calculations for these structures.

## DESCRIPTION OF STRUCTURES

The elevated structures at the four test locations are generally constructed of steel or wrought-iron longitudinal stringers spanning 40 to 60 ft to transverse support bents. Two stringers support each open-deck standard-gauge track on timber ties. The ties also support the electric "third" rail to power the train cars. The support bents carry two active tracks and sometimes a third inactive track (Figures 1 through 4). The two stringers supporting each track are laterally braced between their top flanges and contain either vertical cross bracing at span third points (Figure 3) or lateral bracing between the bottom flanges. The stringer pairs supporting each track are generally connected with a top and bottom flange horizontal strut at the span third points (Figure 3).

Each of the four line segments was constructed under a separate contract between 1891 and 1905. Although the various line segments are of a similar type of structural system, there were significant variations in design and construction details among the segments.

The South Mainline structure was built in two phases. The present southbound structure was built in 1891 using wrought iron. The present northbound structure was added in 1905 and constructed of steel (Figure 1). The stringers are riveted built-up plate-and-angle I-sections connected to the support bents with full-depth web angles (Figure 5). A stringer expansion seat bearing connection ("pocket") is typically lo-

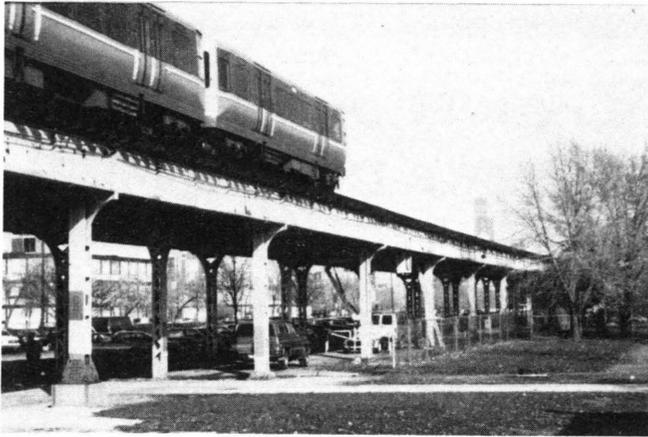


FIGURE 1 South Mainline elevated structure.



FIGURE 3 View from below Englewood test structure.



FIGURE 2 South Loop elevated structure.



FIGURE 4 Jackson Park elevated structure.

cated on one side of every third bent. The support bent columns are located below the centerline of each track (similar to Figure 3). Longitudinal forces on the elevated structure are resisted by bending of the columns.

The Englewood Line structure was built in 1905 using "railway bridge steel." The construction is very similar to that of the northbound South Mainline structure, except for the addition of longitudinal vertical x-bracing at every fourth bay (Figures 3, 5, and 6). The x-bracing, or "traction bracing," resists longitudinal forces applied to the elevated structure. A stringer expansion seat-bearing connection typically is located on one side of every fourth support bent.

The South Loop Line structure in the test area was built in 1891 using wrought iron (Figure 2). The stringers and end connections are similar to those of the South Mainline structure. The support bent columns are located outboard of the two supported tracks, forming a rigid frame for supporting the track stringer gravity loads. The transverse ("cross") girders are single-web riveted built-up I-sections. Longitudinal

forces applied to the track by train braking and accelerating are resisted by cross-girder horizontal bending and longitudinal column bending.

The Jackson Park Line was constructed in 1892 using "soft steel" (Figure 4). The stringers were replaced in 1940, and the stringer end connections were modified in 1964. The stringers are riveted open-web trusses with chords and web members composed of wide-flange tee sections. The stringers are connected to the support bents with a combination of a top chord bearing connection and full-depth web angles (Figure 5). There are no stringer-end longitudinal expansion joints in the structure. The support bent columns are located outboard of the tracks and support the cross girders that span across 63rd Street from curb to curb. The support bent cross girders are double trusses tied with top horizontal batten plates and vertical diaphragms. Again, longitudinal forces applied to the track by train braking and accelerating are resisted by cross-girder horizontal bending and longitudinal column bending.

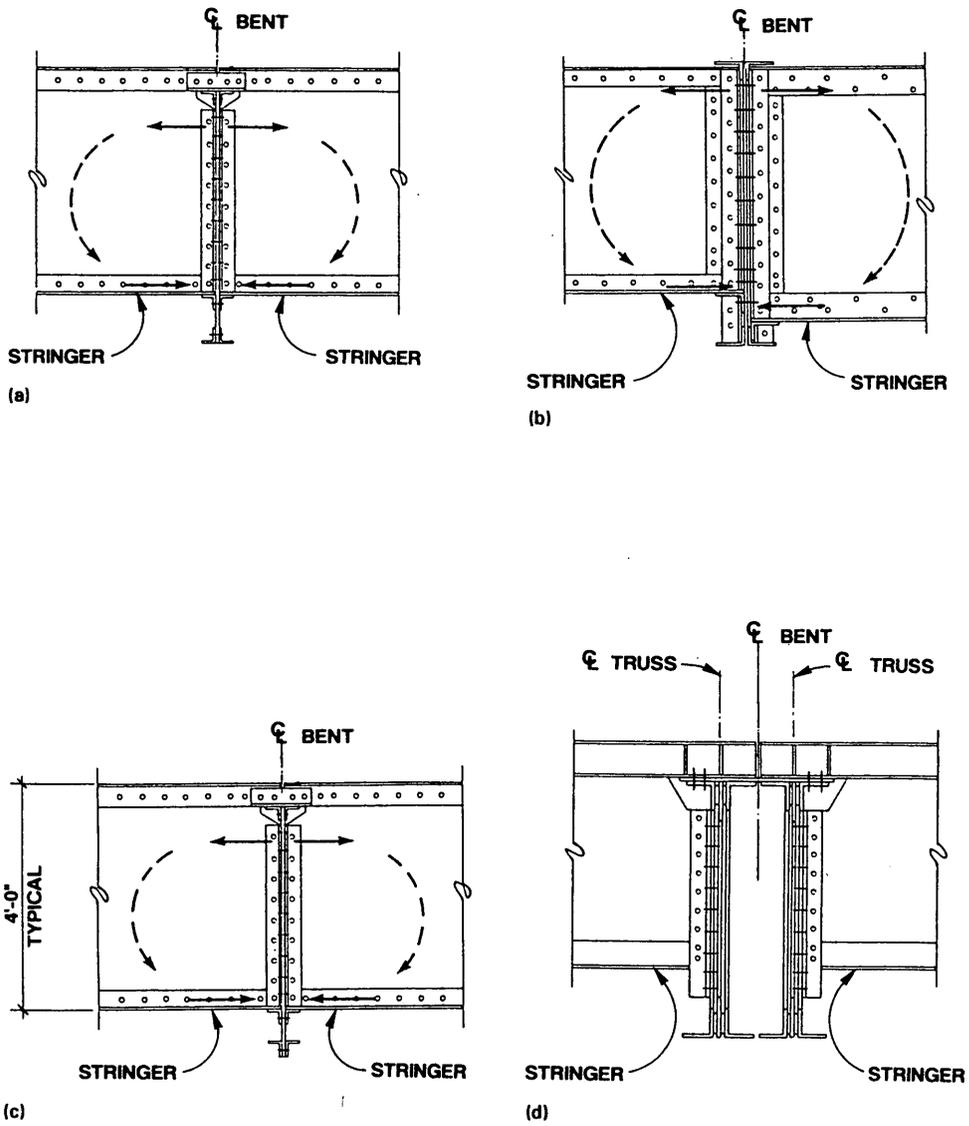


FIGURE 5 Typical details at fixed connections between stringers and support bent: *a*, South Mainline; *b*, South Loop Line; *c*, Englewood Line; *d*, Jackson Park Line.

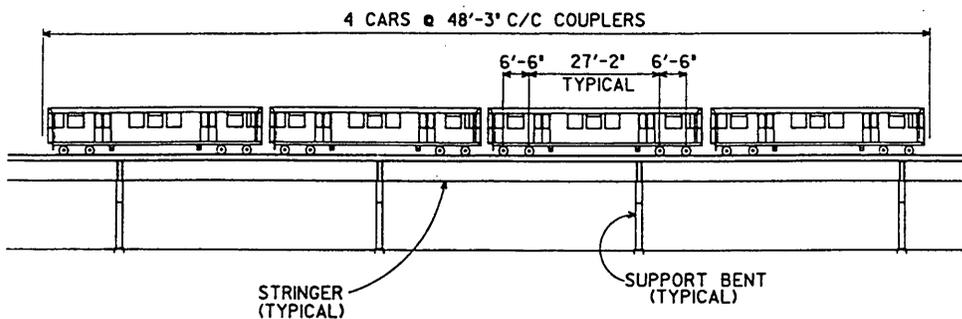


FIGURE 6 Elevation of four-car test train.

**DESCRIPTION OF TESTING PROGRAM**

**Test Type and Loading**

For each line segment test structure, the following types of load tests were performed: (a) static tests, (b) dynamic tests at controlled speeds, (c) train braking tests, and (d) rush-hour tests.

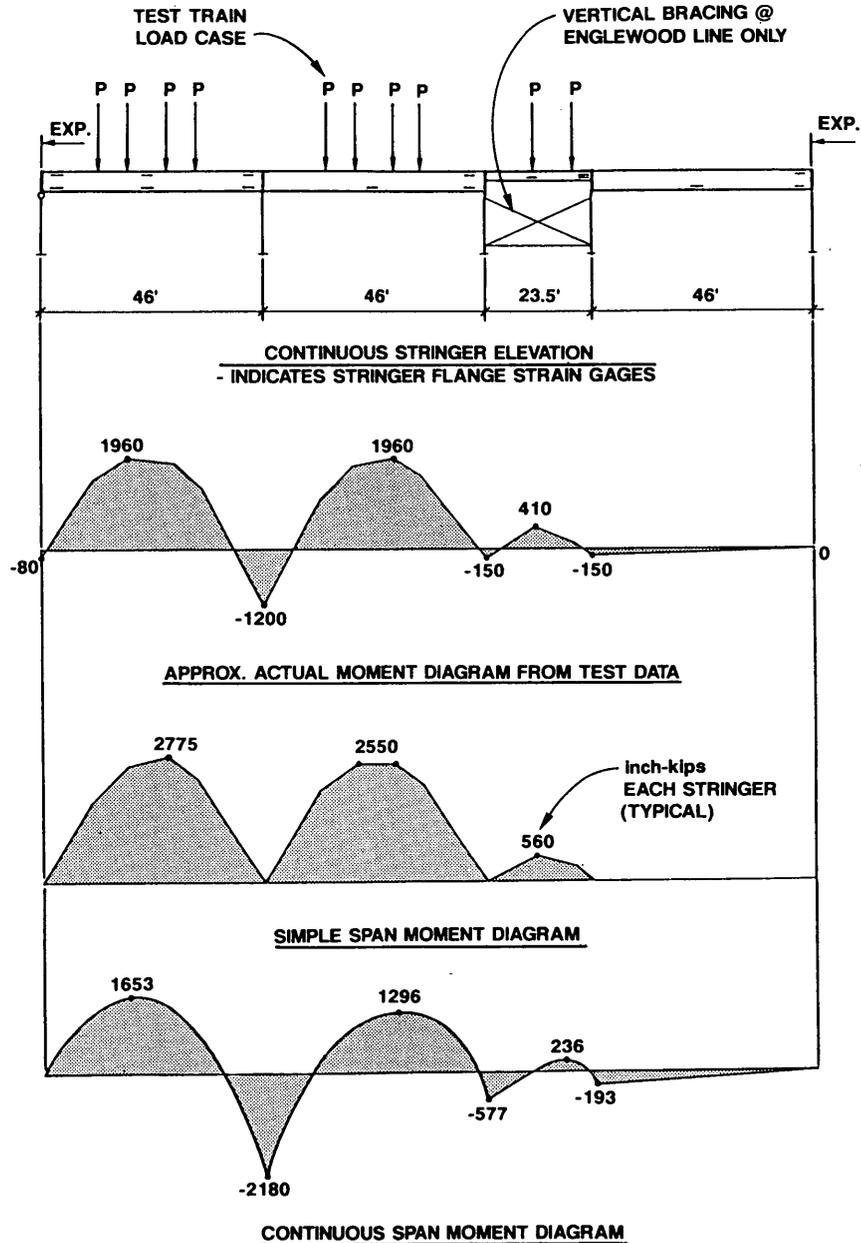
*Test Trains*

An empty four-car test train was furnished by CTA for each test location (Figure 7). Controlled static and dynamic tests

were conducted using the test train on four Sundays during low passenger travel periods. A single-track outage, for 1/4 to 1/2 mi in each direction from the test location, was provided by CTA for test train movement during the testing.

Empty train car weights are approximately 54,000 lb each, or approximately 14 kips per axle for the Series 2601-3200 cars used on the test structures. Passenger loading used by CTA for fatigue rating is 5.5 kips per axle for a total of 19.5 kips per axle.

Static load tests were conducted by stopping the test train at specific locations along the test spans to measure the response at each location. Dynamic tests were performed by moving the test train across the test spans at a crawl speed



**FIGURE 7** Example of comparison of measured and calculated stringer bending moments.

and at various operating speeds up to a maximum speed of 55 mph to measure the effects of impact loading. Dynamic train braking tests were performed to determine the magnitude of longitudinal force caused by the application of emergency brakes (includes track brake). At each test location, braking tests were carried out by stopping the train directly over the test bent and also by stopping the train at various distances from the test bent along the track in each direction.

### Rush-Hour Trains

Rush-hour dynamic load tests were conducted to monitor strains and to evaluate the effect of passenger loading. The measured strain responses included the combined dynamic effects of the eight-car trains moving at normal speeds with passenger loading. Strains in the stringer bottom flanges at midspan were measured for both the inbound and outbound tracks. Since the passengers predominantly travel in one direction during each rush hour, both tracks were monitored to record responses to both the loaded car trains and near-empty car trains.

### Instrumentation

A total of 32 to 42 weldable electric-resistance strain gauges were installed on each test structure.

The gauges were located at the top and bottom flanges of the stringers at midspan and near each end to measure positive and negative bending and axial stresses. The gauges near the stringer ends were positioned to avoid discontinuities adjacent to the end connections and also shear lag in transferring flexural stress to the stringer flange angles from any longitudinal continuity through the end connection components. Strain gauges were installed on the bottom flanges at midspan of

some stringers to monitor positive bending tensile stresses only at these locations.

The support bent cross girders were instrumented with four strain gauges at the point of maximum positive moment to measure vertical and horizontal bending stresses. Strain gauges were installed on support bent columns to measure transverse and longitudinal bending along with axial stresses. The longitudinal vertical x-bracing of the Englewood Line structure was instrumented with strain gauges to measure longitudinal forces caused by the train braking.

The data acquisition system, used to record and process the test data, was composed of a digital scanner, strain gauge completion boards, DC power supply, amplifier, and a portable computer. A digital sampling rate of 500 samples per second per channel was typically used for most of the dynamic tests. A sampling rate of 100 samples per second per channel was used for the dynamic train braking tests and for all of the dynamic tests conducted on the South Loop Line. Severe radio interference, probably caused by the nearby Chicago Central Police headquarters, precluded the use of the more sensitive equipment capable of higher data sampling rates at the South Loop location.

The data obtained from each test were reviewed in the field for accuracy and correctness, using computer analysis and screen plots, before proceeding with the next test.

## ANALYSIS OF TEST RESULTS

### Stringer End Continuity

The results of the static tests are compared to calculated stringer midspan bending stresses, assuming each stringer span is simply supported (Table 1). The comparison, by test structure and span, indicates that midspan bending stresses were 36 to 93 percent of the calculated stresses, assuming simply sup-

TABLE 1 Static Tests (Using Empty Four-Car Test Train): Stringer Bottom Flange Stress at Center of Span

LOCATION	SPAN (FT.)	END SUPPORT CONDITION <sup>a</sup>	STATIC LIVE LOAD STRESS (KSI)		MEAS. CALC. RATIO
			Maximum Measured	Calculated Simple Span <sup>b</sup>	
Jackson Park	50	Fix-Fix	4.2	4.5	0.93
Englewood	46	Exp-Fix	2.5	3.6	0.70
	46	Fix-Fix	2.5	3.6	0.70
	23.5	Fix-Fix	2.1	3.4	0.62
	46	Fix-Exp	3.0	3.6	0.83
South Loop	40	Exp-Fix	2.3	4.0	0.57
	39.8	Fix-Fix	1.4	4.0	0.36
	60	Fix-Exp	3.1	4.1	0.77
South Mainline	46	Exp-Fix	2.9	3.5	0.85
	46	Fix-Fix	2.8	3.6	0.80
	46.8	Fix-Exp	3.1	3.6	0.85

<sup>a</sup> Fix = Full-Depth Shear Angle Connection

Exp = Expansion Seat Connection

<sup>b</sup> Bending Stress Assuming Span Simply-Supported

ported spans. Additional data analysis will show that, except for the Jackson Park structure, the reduced midspan stresses were the result of partial continuity at the stringer-end support connections.

The stringer-end support connections consist of full-depth web connection angles (Figure 5) except at the expansion seat bearing connections, typically located at every third or fourth bent support. The partial continuity, provided by the full-depth web connections on both sides of a support bent, transfers stringer negative bending moment across the support bent and into the adjacent stringer span. This continuity, or stringer end rotation restraint, causes tension through the connection angle top rivets and compression or thrusting through the bottom portion of the connection angles or bottom flange ends. The rotational stiffness of these particular connections is enhanced by the near full-depth length of the connection angles, the short outstanding legs and rivet gauges of the connection angles, and the opposing stringer end rotations caused by the train axle loads acting simultaneously on both spans.

The degree of stringer end continuity is determined for these test structures by comparing measured static load end moments with calculated end moments, assuming the stringers are continuous over, and fixed to, the columns or cross girder. Bending moment diagrams for one Englewood test structure static load case are shown in Figure 7. This figure shows the approximate measured bending moments calculated from the test data. The measured axial stresses, sometimes caused by higher compressive forces across the bottom of the end connections than the top tensile forces, or by frozen expansion seat connections, are separated from the measured bending stresses. The bending moments calculated at the strain gauge locations from the bending stresses are extrapolated to the support bent centerlines using calculated simple span or continuous span shears.

Comparisons between measured end moments and calculated continuous span end moments are summarized in Table 2. The continuity, expressed as a percent ratio of measured moment to calculated continuous span moment, varied from 25 to 86 percent. The percent degree of continuity compared very well to the measured reduction in midspan stresses shown in Table 1.

The maximum continuity (72 to 86 percent) was found in the South Loop Line, which has the longest vertical end connection angles relative to the depth of the stringer (Figure 5). The Englewood and South Mainline structures indicated less continuity (25 to 84 percent) because of the web coped below the top flange angles (Figure 5). The top flange tie angles are deteriorated and too small to increase the degree of stringer continuity at these structures. The static tests on the Jackson Park structure indicated very little or no measured end continuity. The double truss cross-bent support at this structure (Figure 5) allowed nearly free end rotations of the stringers through twisting of each cross truss between vertical diaphragm spacers.

### Cross Bent Twist

The results of the static tests for the cross-girder bottom flange stresses were compared with calculated stresses using rigid frame analysis. The measured vertical bending stress for the Jackson Park structure compared very well with the calculated stress. Lower calculated vertical bending stresses than measured stresses for the South Loop structure indicated a lack of full fixity at the cross girder to column connections.

Horizontal bending stresses in the cross-girder bottom flanges were measured for both the Jackson Park and South Loop structures. This increased the total measured stress in the girder flange outstanding legs by 60 and 47 percent, respectively, over the measured vertical stresses. The horizontal flange bending is a twisting behavior of the cross girders caused by end rotation of the stringers. The twisting of the South Loop cross girder appears to be caused by the rotation of the near fully continuous stringers at a support between unequal adjacent spans (39.8 ft and 60 ft).

### Impact (Dynamic Amplification)

Dynamic tests, using the empty four-car test train, were conducted to measure the effects of trains moving along the elevated structures at various speeds. The results of the stringer dynamic tests are summarized in Table 3. The dynamic amplification of stress caused by moving live loads, or "impact,"

TABLE 2 Static Tests (Using Empty Four-Car Test Train): Stringer Bending Moment Continuity at Supports

LOCATION	ADJACENT SPANS (FT.)	MEAS. MOMENT <sup>a</sup> CALC. CONTINUOUS <sup>b</sup> RATIO
Jackson Park	50 - 50	0
Englewood	46 - 46	0.45 - 0.62
	46 - 23.5	0.26 - 0.84
	23.5 - 46	0.35 - 0.46
South Loop	40 - 39.8	0.86
	39.8 - 60	0.72 - 0.81
South Mainline	46 - 46	0.25 - 0.34

<sup>a</sup> Average measured moment in both stringers with each adjacent span loaded.

<sup>b</sup> Calculated continuous span moments are the theoretical calculated moments assuming the stringer is continuous over, and fixed to, the columns or cross-girder, as determined by computer frame analysis.

**TABLE 3 Dynamic Tests (Using Empty Four-Car Test Train): Stringer Bending Vertical Impact**

LOCATION	SPAN (FT.)	SPEED (MPH)	PERCENT IMPACT <sup>a</sup>		
			Average <sup>b</sup> Measured	Maximum <sup>b</sup> Measured	AREA Design
Jackson Park	50	23 - 33	14	18	55
		46 - 55	19	23	55
Englewood	46	31	26	34	56
		41 - 48	27	39	56
	23.5	31	33	34	59
		41-48	34	46	59
South Loop	60	24 - 33	7	15	53
		44	11	16	53
South Mainline	46	31 - 38	22	30	56
		44 - 51	27	40	56

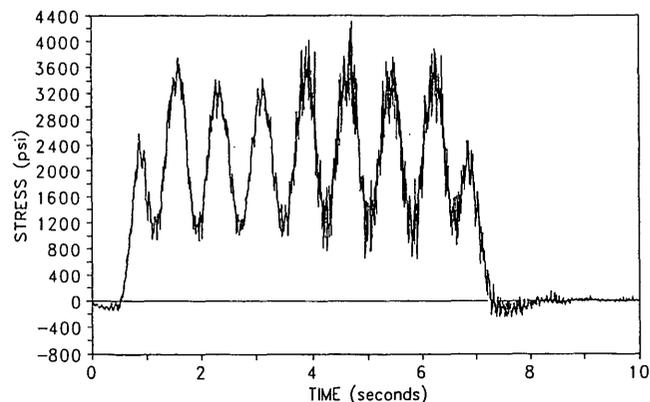
<sup>a</sup> Impact = Percent increase in peak stringer bending stress, at mid-span, from the moving test train loading across a given stringer, compared to the peak measured static load stress for that stringer.

<sup>b</sup> One impact event = maximum recorded impact under one passage of entire test train.

is herein defined as the percent increase in peak measured stress from the moving test train loading compared with the peak *measured* stress at the same location from the static test train loading.

The passage of a moving train across a stringer span produces a stress-time history record of a variable amplitude complex stress cycle. An example of a measured dynamic stress record is shown in Figure 8. The stringer midspan bottom flange stress cycle indicates the passage of an eight-car rush-hour train (Figure 8). Each group of car axles crossing the span produces a variation (subcycle) in stress as shown by the nine peaks. The dynamic effects of the moving train load are shown by the higher-frequency oscillations in the stresses, or vibrations, with frequencies typically in the range of 70 to 100 Hz for these elevated structures.

The measured impact loading for these structures is compared with the AREA manual (3) design impact values in Table 3. The average measured impact loading for the stringers varied from 7 to 34 percent. Although the magnitude of impact varied between test locations, the average measured impact values varied inversely with the stringer span length



**FIGURE 8 South mainline stringer dynamic response at midspan of bottom flange for eight-car rush-hour train (speed, 44 mph).**

(Table 3). There was some general increase in the measured impact values at higher train speeds. However, in some cases, the effect of increased train speed was negligible. The maximum measured impact of 46 percent was recorded for a short 23.5-ft span.

#### Fatigue Assessment Data

The results of the rush-hour dynamic tests conducted on the stringers of each of the four test structures are summarized in Table 4, presenting the average and maximum measured stresses in the stringer bottom flanges at midspan for rush-hour trains traveling in each direction. These measured stresses include the combined effects of the eight-car trains moving at normal speeds with typical passenger loading.

A comparison of the measured stringer stresses for the eight-car rush-hour trains with the dynamic stresses for the empty four-car test trains indicates approximately the same stresses occurring in the Jackson Park and Englewood test structures. The rush-hour trains were almost empty of passengers when they passed these test locations.

A comparison of the average measured peak stress for the South Loop and South Mainline rush-hour tests with the dynamic empty train tests indicates a 14 and 11 percent average stress increase primarily caused by the weight of passengers. The maximum recorded increase in stress over the average empty train tests was 25 and 26 percent, respectively.

The rush-hour average response data (Table 4) and the actual variable stress cycle plots (Figure 8) can be used for fatigue life assessment of the stringer bottom flanges at midspan. These data verify that the train loads of 19.5 kips per axle (plus impact), used by the CTA for fatigue evaluation, are conservative for these test structures. The average weight of passengers during the rush-hour tests was approximately 1.5 to 2 kips per axle at the South Loop and South Mainline test locations and was negligible at the Jackson Park and Englewood test locations.

The dynamic test results indicate that the partially continuous stringer end connections undergo variable stresses with

**TABLE 4** Rush-Hour Dynamic Monitoring (Using Eight-Car Trains with Passengers): Stringer Bottom Flange Stress at Center of Span

LOCATION	SPAN (FT.)	SPEED (MPH)	LIVE LOAD PEAK STRESS (KSI)		
			Prevailing Commuter Travel Direction		Opposite Direction
			Ave. Meas. <sup>a</sup>	Max. Meas. <sup>a</sup>	Ave. Meas. <sup>a</sup>
Jackson Park	50	20 - 48	4.7	5.4	4.1
Englewood	46	38 - 44	2.9	3.5	2.7
	23.5	38 - 44	2.9	3.6	2.6
South Loop	40	15*	2.8	2.9	-
	39.8	13 - 15*	-	-	1.6
	60	13 - 15*	3.5	3.9	2.8
South Mainline	46	42 - 44	4.0	4.5	3.7

\* Train speed restriction at time of test monitoring.

<sup>a</sup> One peak stress event = maximum recorded stress under one passage of entire train.

the passage of each train car. The associated tension forces through the top portion of the connections reduces fatigue life of the web connection angles and top rivets. The design and behavior of the stringer web-angle end connections are currently under evaluation at each structure using the degree of continuity measured by this test program, as summarized in Table 2.

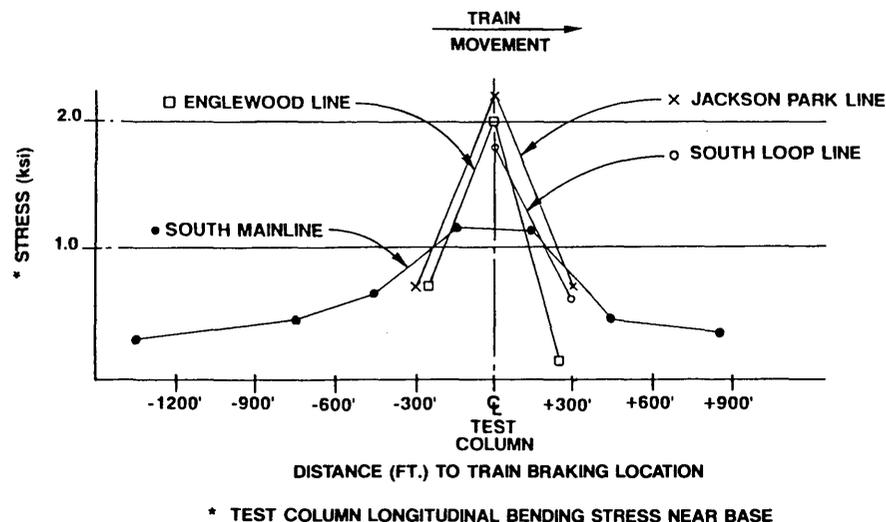
#### Train Braking Effects

The results of the train braking tests on the four structures are summarized in Figure 9. Longitudinal forces were computed on the basis of measured bending stresses near the test column bases or axial stresses in vertical bracing. The Figure 9 graph, showing the maximum test column bending stress relative to the distance of the braking train from the test column, indicates that the longitudinal braking forces are non-uniformly distributed along the track structure, and the rails are effective in distributing those braking forces across lon-

gitudinal structural expansion joints. Longitudinal structural expansion joints are typically located 140 to 160 ft center to center.

The Jackson Park and South Loop test locations recorded a reduction of column bending stress 300 ft away from the train braking location to about one-third of the maximum value directly below the braking train. The measured maximum longitudinal forces in the column directly below the test train were approximately 2.4 and 1.6 kips, respectively, for the Jackson Park and South Loop braking tests. The AREA manual longitudinal design load for both structures is approximately 3.8 kips in each column, distributed uniformly along a 200-ft length of elevated structure directly below the 200-ft-long test train. The AREA load is calculated as 15 percent of the test train load (weight) multiplied by a standard factor of 0.80 to account for continuous rail along the elevated structure exceeding 1,200 ft in length.

The Englewood Line segment was built with vertical longitudinal x-bracing at every fourth bay, as shown in Figure 7.



**FIGURE 9** Measured effects of longitudinal force (braking) tests using four-car test train.

For the test train stopping 250 ft away from the x-bracing, the reduction in x-bracing forces and column bending stresses was approximately one-half, or less, of the maximum recorded braking force. The maximum horizontal force resisted by the vertical bracing was 22.6 kips, and by column bending 0.9 kips, when the train was stopped directly over the test spans. The AREA longitudinal design load for the entire test train is 25.9 kips distributed to 200 ft of elevated structure, consisting of one bay of vertical x-bracing and five columns in bending.

The dynamic braking tests conducted for the South Mainline structure included stopping the test train using the emergency brakes at seven locations along the elevated structure up to a distance of 1,350 ft away from the test column. Test records indicated that the longitudinal braking forces were nonuniformly resisted up to a distance of at least 850 ft away from the braking test train, resulting in a distribution of horizontal load over a length of elevated structure exceeding 1,700 ft. Structural rebound and longitudinal oscillation (rocking) were shown by the stress record at 1,350 ft from the just-stopped test train. The record indicated very little column bending during train braking, but that significant column stresses, equal to 25 percent of the maximum recorded column stress, occurred at this distance immediately after the train had stopped. The maximum longitudinal force in one column introduced by braking was measured to be approximately 3.6 kips. This value is compared with the AREA design load of approximately 5.3 kips per column, distributed along 200 ft of the elevated structure directly below the 200-ft-long test train.

## CONCLUSIONS

Diagnostic load tests have provided knowledge about the actual response of the elevated steel transit structures to operating loads. This knowledge, currently being used to supplement the engineering analysis and rating calculations and to design rehabilitations for these structures, is summarized as follows:

1. Measured stringer midspan bending moments were 25 to 86 percent of those calculated, assuming simply supported spans, because of partial continuity through the stringer end support connections. The full-depth web-shear framing angle connections exhibited a wide variation in degree of continuity depending on the particular connection configuration. One test structure developed up to 86 percent of the theoretical end moment determined from a fully continuous analysis.

2. Stringer end rotations can impose out-of-plane stresses in cross-girder supports. Cross-girder horizontal bending stresses increased the total measured stress at one test location by 60 percent more than the vertical bending stresses. The total measured stresses were well within design allowable stress values.

3. Vertical impact testing was carried out using empty test trains. Measured vertical impact loading on the stringers, expressed as a percent of measured static stresses, averaged 7 to 34 percent for the test spans of 23.5 ft through 60 ft. The average measured impact varied inversely with the span length. A maximum vertical impact of 46 percent was recorded for

the short 23.5-ft span. There was no appreciable correlation between impact and train speed.

4. Stringer midspan bending stresses were monitored during rush-hour commuter travel periods. The measured stresses indicated that the standard CTA fatigue design load of 19.5 kips (plus impact) per axle is conservative for the current trains in service and may therefore be safely used to estimate the remaining fatigue life. The actual variable amplitude stress plots, recorded during rush hour, can be used for a more accurate assessment of stringer fatigue life.

5. The measured partial continuity provided by the stringer web-shear framing angle connections is beneficial in reducing the stringer midspan stresses. This continuity reduces the fatigue life of the end connections by subjecting the top portion of the connections to variable tensile stresses with the passage of each train car. Retrofitted end connections are being considered to either reduce the continuity or provide improved fatigue resistance for the degree of continuity measured by this test program.

6. The timber open-deck track structure and steel rails were effective in distributing longitudinal forces caused by train braking across longitudinal expansion joints in the elevated structures.

7. Braking tests indicated that the longitudinal forces are nonuniformly distributed along the elevated structure. The typical elevated structure resists the train braking forces through column bending. The measured individual column longitudinal forces were less than AREA design loads uniformly distributed to columns directly below the 200-ft-long braking test train. The measured longitudinal forces significantly decreased in magnitude at a distance of 300 ft from the train. Longitudinal braking forces were nonuniformly distributed over a length of elevated structure exceeding 1,700 ft at one test location.

## ACKNOWLEDGMENT

The authors appreciate the cooperation and assistance provided by the Chicago Transit Authority in conducting the load tests. Also, the authors wish to thank K. R. Hoigard, C. J. Malfitano, and B. J. Smith for their valuable contribution during the instrumentation and data acquisition phases of the test program.

## REFERENCES

1. S. G. Pinjarkar, O. C. Guedelhoefer, B. J. Smith, and R. W. Kritzler. *NCHRP Research Project 12-28(13): Nondestructive Load Testing for Bridge Evaluation and Rating*. TRB, National Research Council, Washington, D.C., Feb. 1990.
2. S. G. Pinjarkar, R. W. Kritzler, R. A. Rolsing, and P. O. McCarthy. *Evaluation and Load Testing of a 100-Year-Old Elevated Steel Transit Structure*. Presented at 3rd Bridge Engineering Conference of the Transportation Research Board and FHWA, Denver, Colo., March 10-13, 1991.
3. *Manual for Railway Engineering, 1990-1991*. American Railway Engineering Association, Washington, D.C.

*The opinions and conclusions expressed or implied in the paper are those of the authors only and are not approved or endorsed by the Chicago Transit Authority.*

*Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.*