

Construction, Monitoring and Evaluation of a Pedestrian Arch Bridge with a Precast Deck System

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ABSTRACT

This paper presents the rationale for, and results of, structural monitoring of two steel basket-handle arch pedestrian bridges which were constructed in stages to minimize traffic disruptions. These bridges utilize a hanger system consisting of high-strength threaded steel rods to support the precast, post-tensioned concrete deck system.

The research team used externally-mounted accelerometers to monitor the free vibration of each hanger during all incremental construction stages as well as one year after completion. The acceleration records for each hanger were analyzed using a fast fourier transform (FFT) to calculate the natural frequency of several modes of vibration. These natural frequencies were then used to estimate the tensile force in each hanger considering both taut-string theory and the flexural stiffness of the steel hanger rod.

The results of this work show the value of structural monitoring during construction and indicate that some moderate changes in structural behavior occurred after the bridge entered service.

BACKGROUND

Introduction

In the summer of 2003, the Iowa Department of Transportation (Iowa DOT) constructed a 262 ft (80 m) span, basket-handle arch pedestrian bridge over Interstate 235 (I-235) near the Botanical Center in Des Moines, Iowa. This bridge, later named the Edna M. Griffin Memorial Bridge, was the first of three nearly identical bridges to be constructed as part of the reconstruction of I-235.

Interstate 235 is the major transportation corridor through the Des Moines metropolitan area and carries significantly more traffic than any other corridor in the state of Iowa. The Iowa DOT is currently rebuilding the entire freeway to current design standards, which will improve capacity and greatly reduce the number of traffic accidents. Construction is scheduled for completion in 2007.

As part of this reconstruction, the Iowa DOT and the City of Des Moines wanted to create a set of signature bridges that would provide a “gateway” into the city. Three basket-handle arch pedestrian bridges were designed to provide a dramatic visual entryway into the state capitol (see Figure 1). The bridges are the centerpiece of the master aesthetic plan that was developed early in the planning for the I-235 project.

Rationale for arch bridges and construction process

The basket-handle arch spans were designed in response to the Des Moines community’s strong pride and association with its arch river bridges as symbolized in the city’s graphic logo. A significant advantage to the basket-handle bridge configuration is the ability to span the entire freeway without a central pier, providing an airy, open feeling, while allowing for a secure pedestrian crossing near four different schools.

The steel arch ribs for each bridge were erected in a single overnight closure of the freeway. In order to accelerate the assembly to the greatest extent possible, each set of ribs was pre-assembled into three units that could be erected in place without the use of falsework.

The precast, post-tensioned deck system was selected to allow construction without the need for falsework near the freeway. From a practical standpoint, this type of bridge would greatly limit the amount of interference to traffic on I-235. Deck panels were installed during strictly limited overnight closures of the freeway when installed near the center of the arch. Once a sufficient number of deck panels had been installed to avoid construction interference with open traffic lanes, installation operations were performed during daylight hours.

Need for Structural Monitoring

The bridges utilize a precast, post-tensioned concrete deck system and represent the first application of precast, segmental construction on the Iowa highway system. The match-cast concrete deck panels are suspended from the steel arch ribs using 36 mm (1.42 inch) diameter high strength steel rods.

In the months following the construction of the Griffin bridge, a considerable amount of cracking of the concrete panels was observed at locations near the more-heavily loaded hanger rods. The assessment of hanger loads during construction of the Griffin bridge was very subjective and consisted only of the contractor and inspection personnel manually “shaking” each hanger to determine relative degrees of tension.

The unusual nature of the bridge structural system, combined with the observed cracking and the upcoming design and construction of two additional basket-handle arch bridges compelled the Iowa DOT to host a post-construction review meeting in February 2004. One of the significant conclusions drawn from this meeting was that a more precise method of determining hanger loads, both during and after construction, of all three bridges was necessary.

The Iowa State University Bridge Engineering Center was contracted to monitor and evaluate the structural performance of the hanger rods on two similar pedestrian bridges which were constructed near 40th and 44th Streets in Des Moines during the summer of 2005. This paper presents results of this study as well as subsequent monitoring which was performed during July 2006.



a) Overall view of bridge looking east



b) Sand-filled barrels were used as ballast during the erection of 40th Street Bridge panels

FIGURE 1 40th Street Pedestrian Bridge over I-235 – Des Moines, IA.

Bridge Description

The arch ribs consist of tubular steel sections which are tapered in both the horizontal and vertical directions from 19.6 in x 27.6 in (500 mm x 700 mm) at the crown to 29.5 in x 49.2 in (750 mm x 1250 mm) at the base. The arch ribs are inclined at approximately 10 degrees and are connected at the crown by a bolted diaphragm. No other lateral bracing is provided other than a tubular steel strut below the deck level at the 44th Street and Griffin bridges.

The precast concrete deck panels were fabricated using a match-casting system in which the first panel was used as the end form during the casting of the second, or adjacent, panel. During this process, the final shape of the deck profile was precisely replicated by providing a very slight angle change at the joint between adjacent concrete panels. In order to avoid placing undue stress on the concrete panels, they must then be supported at this exact same relative elevation when erected and post-tensioned together in the field.

The hanger system used for the bridges consists of a 1.42 inch (36 mm) diameter high strength rod surrounded by a 2.56 inch (65 mm) diameter High Density Polyethylene (HDPE) pipe sheathing. The annular space surrounding the hanger rod was filled with a cementitious grout prior to delivery to the project site. This pre-grouted system required the contractor to be particularly careful in the way the hangers were handled and erected so as to avoid cracking of the grout material. The hangers were anchored in the arch rib using a standard anchor plate and nut. The hangers passed through a 3.15 inch (80 mm) diameter galvanized trumpet pipe in the deck panels and were anchored using a beveled shim plate in combination with an anchor plate and nut. The contractor was able to rotate this beveled shim plate to accommodate slight irregularities in angular construction.

The support elevations of the concrete panels were controlled by the precise adjustment of the anchor nut located at the bottom of the hanger rod directly beneath the panel. During the construction of the Griffin bridge, it became evident that, in some cases, the hanger rod anchorage elevation, and therefore the tensile load in that particular hanger, varied considerably from that anticipated by the designers. Some minor cracking of the concrete panels was observed at the panel locations near these more-heavily loaded hanger rods.

In order to eliminate this disparity, the contractor performed a series of small adjustments in the final elevation of the panels during additional overnight closures of I-235. These overnight closures required considerable expense to the State for traffic control and law enforcement as well as creating inconvenience and potential danger to both the traveling public and the onsite workers.

Construction Sequence

The 40th Street Bridge contains a total of 22 precast concrete panels, while the 44th Street Bridge contains a total of 20 precast concrete panels. The precast panel layout and numbering system for the two bridges are presented in Figure 2. It should be noted that the panel layout for the Griffin bridge was very similar to that used on the 44th Street Bridge.

The panels were segmentally erected and longitudinally post-tensioned using high-strength threaded steel rods meeting the requirements of ASTM A722. In the segmental erection process, the mating faces of adjacent panels were coated with a specialized epoxy adhesive (Unitex Segmental Adhesive) and the panels were then stressed together using a hollow-core jack. The panels are supported by 36 mm (1.42 inch) diameter, high-strength steel bar hangers. The hangers are connected to the arch rib at anchor points located at 4200 mm (13.78 feet) spaces.

The precast panel cross section is not symmetrical about the longitudinal centerline of the bridge. In order to promote bridge deck drainage, the panels were cast such that the eastern side of the panel is considerably heavier than the western side. Due to this unsymmetrical shape, the deck panels, when erected and supported from the hanger rods, and before the deck system was complete, did not hang directly below the longitudinal centerline of the bridge. In fact, these panels were observed to hang as much as 28 inches west of their intended position. The contractor for the 40th Street bridge used a series of sand-filled barrels as ballast (see Figure 1b) on the west side of the deck panels, which caused the panels to hang much closer to their intended position relative to the bridge centerline.

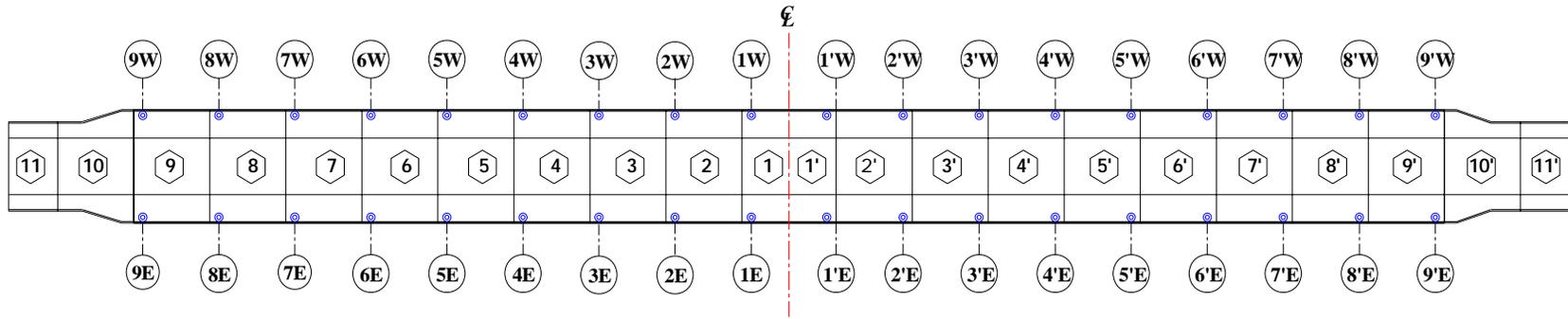
The deck construction sequence for each bridge was virtually identical. A brief summary of the sequence is provided below:

1. The center pair of panels (1 and 1') were glued and stressed together on the ground prior to being lifted using two cranes (one on each side of the arch ribs).
2. The four hanger rods for the center pair of panels (1E, 1W, 1'E and 1'W) were installed and anchored in the arch ribs. The bottom of the hanger rods were held out-of-plumb to match the inclination of the arch ribs.

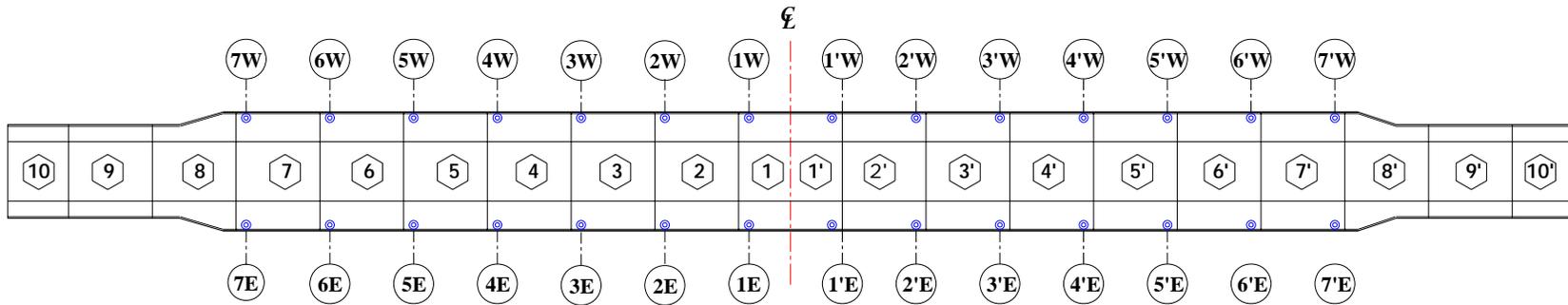
3. The center pair of panels were lifted and held in position just below the bottom of the hanger rods. The panels were carefully lifted as the hanger rods were inserted through the trumpet pipes located near the corner of each panel.
4. Surveyed elevations at the leading end of each panel were measured prior to final release of the panels from the cranes.
5. Each hanger rod was anchored against the bottom of the panels using a pair of beveled washers and nut (Stage 1).
6. Subsequent panels were installed in an alternating sequence (2', 2, 3, 3', 4', 4, etc.) such that the loading on the arch was never more than one panel out of symmetry. Each additional panel added was treated as a separate loading stage for monitoring purposes (e.g. installation of panel 2' = Stage 2, panel 2 = Stage 3, etc.).

At a number of times during the panel erection process, the contractor made intermediate adjustments in the panel support elevations to maintain, to the greatest extent possible, the correct match-cast alignment as well as to roughly equalize the hanger loads. The contractor used a hollow-core jack attached to the bottom of the particular hanger rod to slightly lift panels in question. The anchor nut at the bottom of the panel would then be adjusted as necessary and the jack released to resume normal support on the anchor nut.

During the erection of the deck panels for the 40th Street Bridge, the contractor found that the entire deck was being erected as much as 2 inches (50 mm) higher than the elevation shown in the plans. In order to correct this error, the deck panels were incrementally lowered until the correct elevation was reached. This adjustment operation, as well as all intermediate adjustments, was performed sequentially at alternating hangers so that no panel was out-of-phase from adjacent panels by more than $\frac{1}{4}$ inch (6 mm) at any time. It should be noted that on the very shortest hangers, an adjustment in hanger elevation of this amount creates a change in hanger load of nearly 40 kips (178 kN).



(a) 40th Street Bridge



(b) 44th Street Bridge

FIGURE 2 Precast panel layout – I-235 pedestrian bridges.

MONITORING AND EVALUATION METHODS

Background

The use of vibration monitoring methods for estimating the tension in a bridge stay cable has been used for many years. The typical process is to mount accelerometers on the cables at a convenient location and then use a mechanical shaker is used to provide a reliable means of excitation of the cable (1). A similar technique has been used to assess the tension in external post-tensioning rods in both laboratory and field tests (2).

In ordinary cable monitoring, such as performed on a suspension or stay cable, the simply taut-string equation is typically used. However, the simple theory may cause unacceptable errors in many applications since the cable sag and bending stiffness are ignored (3). Contrary to the process used for strands or wire rope, the vibration monitoring for members such as the referenced hanger rods, must consider both the flexural stiffness as well as the string vibration of the overall system.

Instrumentation, Data Collection and Data Processing

The test equipment used for the field evaluation consisted of an Optim 3415AC Megadac with laptop PC controller running Test Control Software. The accelerometers were mounted to the hangers approximately 5 ft above the deck panels using steel clamps and were oriented with their primary axis perpendicular to the centerline of the bridge.

After placing an accelerometer, the initial values were balanced to allow for stabilization of the sensor. The acceleration record for each hanger was collected by manually exciting the hanger using a suddenly applied, horizontal force approximately 4 ft above deck level. The external force was applied perpendicular to the bridge centerline. Data were collected for a period of approximately 10 sec. from which free vibration records were utilized for data processing. The test was repeated at least two times per hanger to verify the reproducibility of the data. Commercial software was used for processing the free vibration data and to obtain the natural frequencies as illustrated typically in Figure 3.

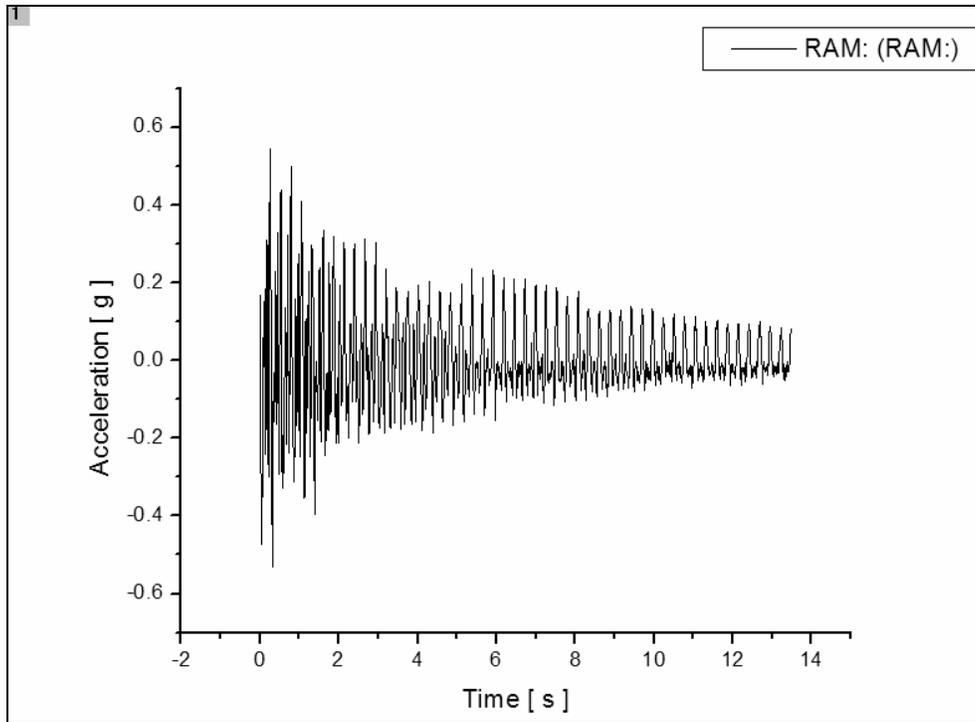
The hanger tensile forces were computed based on vibration methods and cable tension force estimates based on string and beam theory. The following expression was used and the parameters are defined as:

$$T = \rho A \left(\frac{L}{n\pi} \left[\omega_n - (\beta_n L)^2 \sqrt{\frac{EI}{\rho A L^4}} \right] \right)^2 \quad (1)$$

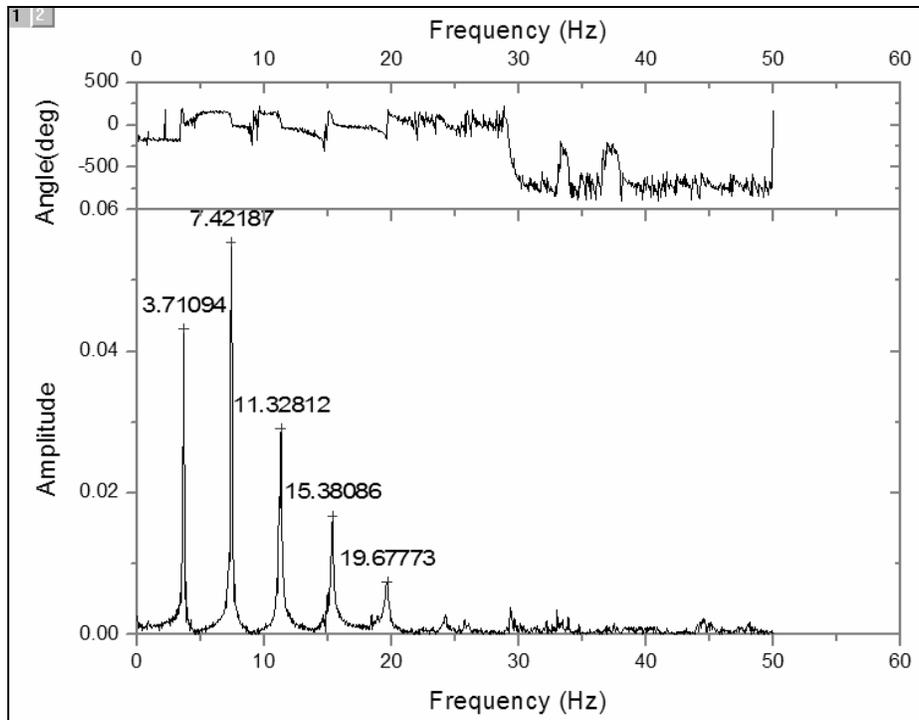
Where:

- T : Tensile hanger force [kip]
- ω_n : Natural frequency obtained from the Fast Fourier Transform (FFP) analysis
- $\beta_n L$: Beam frequency constants associated with member end boundary conditions
- L : Hanger rod length [ft]
- E : Modulus of elasticity of 29,000 ksi for the high-strength steel bar
- A : Section of the high-strength steel rod (1.58 in²)
- I : High-strength steel bar inertia, equal to 9.58 x 10⁻⁶ ft⁴
- ρ : Density corresponding to the cross section, equal to 23.13 lb-sec²/ft⁴ [concrete + steel]
- n : Mode number corresponding to ω_n

The use of equation (1) required some assumptions, partly due to the composite material makeup of the hangers and due to the hanger end support connection details. In particular, that includes the member end support boundary conditions, the effective hanger length, the hanger modulus of elasticity and inertia, etc. The effective flexural stiffness parameter EI of the cross section was computed using only the high strength steel bar properties (i.e., the grout was assumed to be cracked and contribute no flexural stiffness). However, the hanger density was assumed to include both the grout infill plus the steel bar. The length of the hangers was assumed to be the theoretical length between the anchor point in the arch rib and the bottom of the deck panel. For all hanger force computations, the member end support boundary conditions were assumed to be both pinned and fixed against rotation to provide a “bracketed” solution.



a) Real time vibration record



b) FFT Results

FIGURE 3 Measured and processed vibration data for Hanger 2'W of the 40th Street Bridge.

2005 Results; 40th St. Bridge

40th St. Bridge; Staged Panel Placement

Figure 4 presents calculated hanger forces for Construction Stages 1, 2 and 3. The data include force calculation for two member end boundary condition assumptions: 1) complete rotational fixity at both ends (fixed) and 2) no rotational fixity at both ends (pinned). A brief discussion of the data for each stage is described below.

Stage 1 The estimated dead load of the panel (including ballast) was approximately 69.9 kips. The calculated forces for the four hangers was 69.8 kips and 86.4 kips, respectively, for the fixed and the pinned conditions. The four hanger forces were relatively similar and compared closely with the estimated theoretical hanger forces of 17.5 kips (based on equal distribution of dead load to all hangers). See Figure 4a.

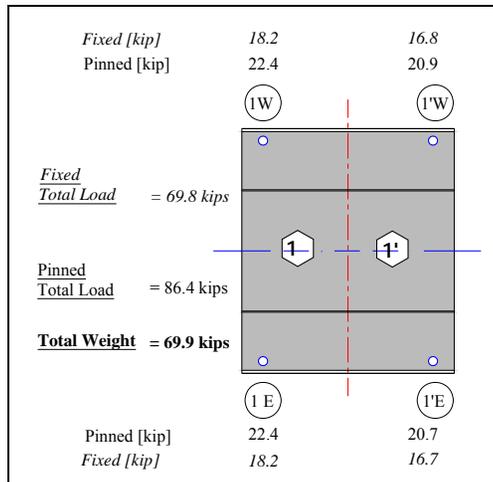
Stage 2 The summation of the six hanger forces at the completion of Stage 2 was 126.4 kips and 153.9 kips, respectively, for the fixed and pinned conditions. This compares with the total estimated dead load of the panels of 126.4 kips. Note that the hanger forces in 1'W and 1E were reduced after the second panel was placed, while the other four hanger forces increased in magnitude (Figure 4b).

Stage 3 After placement of the Stage 3 deck panel, the estimated dead load was approximately 183.0 kips. In comparison, the calculated forces for the six hangers was 181.4 kips and 217.7 kips, respectively, for the fixed and pinned conditions. It is interesting to note that hanger 2W (one of the newly placed hangers) was very lightly loaded compared to expectations (4.0 kips for the fixed condition). In contrast, hanger 2'E carried 36.7 kips for the fixed condition. There was a considerable non-uniform redistribution of hanger forces throughout the three panels at the end of the Stage 3 construction, with significant hanger force unbalances noted at several panel point pairs (e.g. 1W and 1E, and 2'W and 2'E) (Figure 4c).

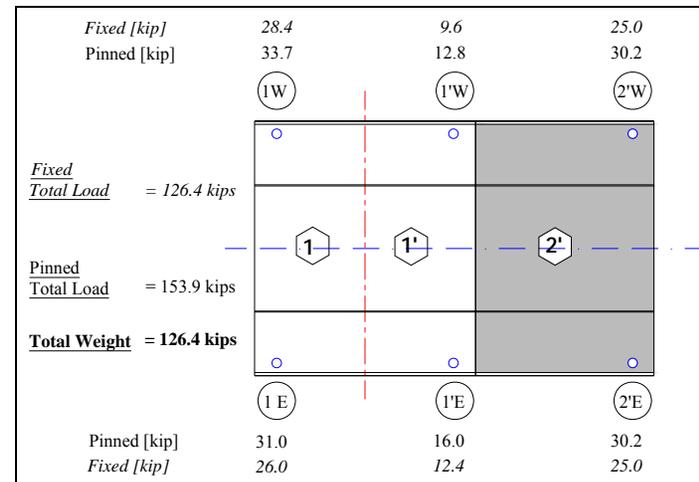
The hanger force results for the staged construction, as well as the discussion above regarding the sensitivity of the results based on assumptions, suggest that the hanger member end boundary conditions contain some degree of fixity; further, the pinned and fixed conditions likely provide a bounded solution for the actual hanger forces with the results likely closer to the fixed condition. In the interest of simplicity, subsequent data presentation in this paper will include data based upon the assumption of member end boundary condition fixity only unless otherwise stated.

To further validate the magnitude of the hanger forces determined as noted above, strain data were also collected during the staged construction. The data are formally not presented in this paper because of some inconsistencies at several hanger locations due to construction handling issues. However, a significant amount of data from undamaged sensors validated that the hanger force data tended more toward the assumption of fixed conditions.

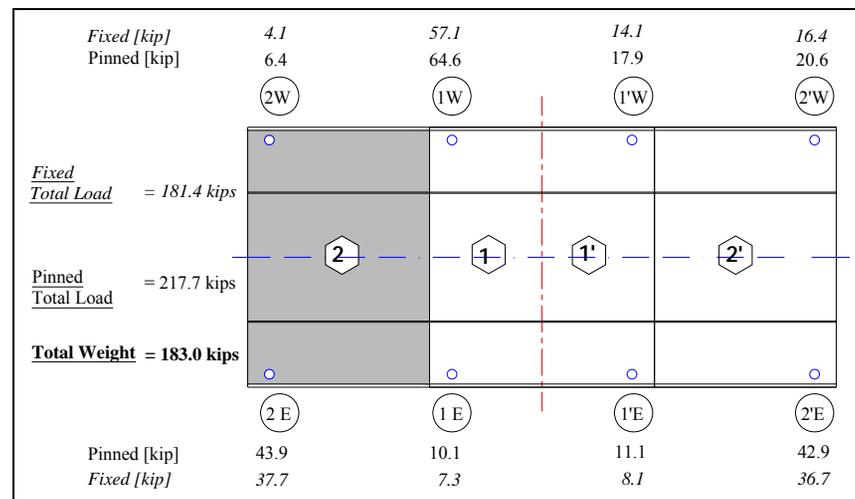
It is also noteworthy that during various phases of the panel placement, visual observation of the hanger near the arch connection indicated localized curvature at numerous locations prior to placement of the deck panels, also suggesting some degree of rotational restraint. This particular condition was observed at numerous deck panel locations. Subsequent sensitivity calculations of hanger force looking at reasonable ranges of parametric values indicated that the boundary condition assumptions provided the greatest sensitivity.



a) Stage 1



b) Stage 2



c) Stage 3

FIGURE 4 Schematic view of hanger forces calculated using free vibration method

40th St. Bridge; Preliminary Completion of Panel Placement

Table 1 presents the measured hanger forces after preliminary placement of all of the bridge panels (i.e., prior to any hanger force adjustment). Since the four end hangers (two each at the ends of the bridge) were too stiff to effectively measure force with the vibration method, hanger force data for these hangers are not included in Table 1. The data shown were collected prior to the placement of the last four end panels (two panels near each abutment). Therefore, the dead load of all of the panels assumed to be supported by the 32 hangers (16 on each arch) was used for comparison.

Based on the assumption above, the estimated deck dead load was approximately 966 kips (this excludes the four end panels, two near each abutment). In comparison, the calculated forces for the hangers (excluding all four end hangers) was 862.4 kips and 1081.1 kips, respectively, for the fixed and pinned conditions.

Other observations noted include the following:

- There was a relative lack of consistency in hanger force values (e.g. 8W, 3W, 1'W, 4'W were extremely low and 1W and 7'W were very high).
- The hanger pairs on the east and west arches tended to be out of balance with each other at many locations (e.g. hangers 5, 1', 3', 4', 5' 7' and 8').
- The total of the hanger forces on the east arch was approximately 7% higher than the forces on the west arch. This is due at least partially to the placement of ballast on the west side of the deck unit to try to balance the unsymmetrical deck weight. In fact, it is possible that this imbalance may have been caused by a slight error in the actual weight or positioning of the ballast barrels (relative to the contractor's calculations).

40th St. Bridge; Final Adjustment of Panel Placement

Table 1 also presents the measured hanger forces after the contractors' adjustment of the hangers following completion of the bridge. Observations noted from the table include the following:

- There was relatively good distribution of hanger forces throughout bridge.
- The total summation of hanger forces was reduced by approximately 40 kips. One possible explanation is that the bridge was constructed approximately 40 mm high (at the end) to 70 mm high (near midspan) and this alignment was corrected by lowering the panel elevations prior to the retesting of the hanger forces. This process might have increased the abutment reactions while decreasing the overall hanger loads.

Figure 5 shows a plot of the hanger forces for both of the arches to illustrate the redistribution of the hanger forces after adjustment. The calculated approximate panel dead load values are also shown for comparison. As noted the adjustments eliminated the outliers relative to the design values and improved the overall distribution of the forces.

TABLE 1 40th Street Bridge hanger forces before and after adjustment

<i>Hanger Designation</i>	<i>Before Adjustment</i>		<i>After Adjustment</i>	
	<i>West Arch [kip]</i>	<i>East Arch [kip]</i>	<i>West Arch [kip]</i>	<i>East Arch [kip]</i>
9	-	-	-	-
8	1.2	0.0	17.7	5.2
7	18.9	12.4	21.9	27.7
6	39.9	54.3	27.5	25.4
5	43.7	13.4	25.8	21.9
4	27.1	40.6	27.4	27.4
3	3.4	3.4	22.5	40.7
2	18.8	33.5	20.9	24.7
1	75.4	50.1	30.7	27.4
1'	3.1	28.5	33	29.6
2'	30.7	30.7	32.2	27.8
3'	54.6	10.5	25.4	19.9
4'	4.4	49.2	23.8	24.6
5'	28.9	2.4	36.4	38.3
6'	24.4	24.4	28.9	31.1
7'	60.8	32.4	23.4	20.4
8'	12.1	28.7	16.8	6.2
9'	-	-	-	-
Total Load	862.1		812.6	
Total Dead Load	996.0		996.0	

Note: "--" denotes data not considered reliable

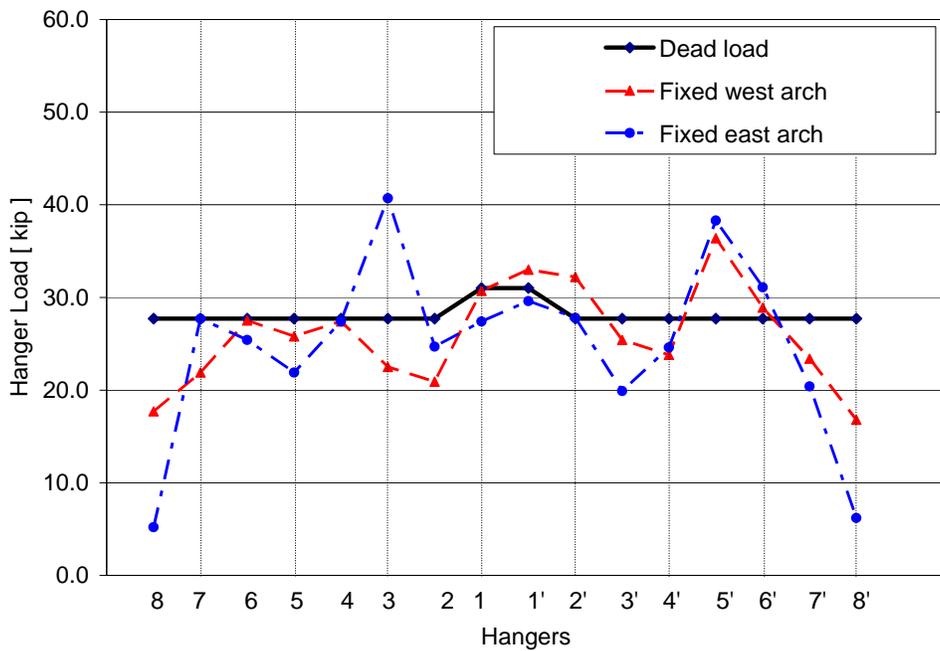


FIGURE 5 Hanger forces for both arches of the 40th Street Bridge after adjustment.

2005 Results; 44th St. Bridge

44th St. Bridge; Preliminary Completion of Panel Placement

Table 2 shows the hanger forces after initial placement of the deck panels (i.e., prior to any hanger force adjustment). As noted earlier in the paper, this bridge has a strut near each abutment that helps support, along with the abutments, some of the panel dead load in the end span region. Since the four end hangers (two each at the ends of the bridge) were too stiff to effectively measure force with the vibration method, force data for these hangers are not included in the Table 2. The dead load of all of the panels assumed to be supported by the 24 hangers (12 on each arch) was used for comparison.

The estimated deck dead load was approximately 742 kips (this excludes the six end panels, three near each abutment). In comparison, the calculated forces for the hangers (excluding all four end hangers) was 605.1 kips and 780.3 kips, respectively, for the fixed and pinned conditions.

The following are some additional observations:

- There was a relative lack of consistency in hanger force values (e.g. 4W and 5'W were very low and 5W very high).
- The hanger pairs on the east and west arches tended to be out of balance with each other at many locations.
- The total of the hanger forces on the east arch was approximately 14% higher than the forces on the west arch. It was noted previously that the deck cross section is not symmetrical; it is a deeper section on the east side, which contributes to the unbalanced forces presented. It should be noted that the design plans for both the 40th and 44th Street bridges indicate equal loading on both the east and west arch rib hangers. However, the unsymmetrical cross-section of the deck panels makes this condition unlikely.

44th St. Bridge; Final Adjustment of Panel Placement

Table 2 also shows the hanger forces after adjustment of the initial hanger forces following completion of the bridge. Observations noted from the table include the following:

- The distribution of the hanger forces was much more uniform throughout the bridge.
- The total summation of hanger forces reduced by 6 or 7 kips. This may have been caused by the hanger adjustments which slightly lowered the elevation of the deck panels near the end of the bridge, causing the strut between the arch ribs to support additional load through the bearings.

Figure 8 shows a plot of the hanger forces for both of the arches to illustrate the redistribution of the hanger forces after adjustment. The calculated approximate panel dead load values for each hanger are also shown for comparison. As noted the adjustments eliminated the outliers relative to the design values and improved the overall distribution of the forces.

TABLE 2 44th Street Bridge hanger forces before and after adjustment

<i>Hanger Designation</i>	<i>Before Adjustment</i>		<i>After Adjustment</i>	
	<i>West Arch [kip]</i>	<i>East Arch [kip]</i>	<i>West Arch [kip]</i>	<i>East Arch [kip]</i>
7	-	-	-	-
6	4.3	34.0	23.7	28.6
5	66.6	10.2	24.3	12.0
4	9.5	11.8	30.0	20.9
3	36.8	18.7	22.2	24.7
2	16.6	42.4	23.4	33.8
1	20.5	39.2	34.0	34.0
1'	39.6	27.9	29.4	23.5
2'	34.9	22.8	24.2	25.6
3'	10.3	17.5	24.7	31.7
4'	22.0	57.1	15.5	37.1
5'	6.6	16.8	16.6	22.6
6'	14.5	24.5	9.1	25.9
7'	-	-	-	-
Total Load	605.1		597.5	
Total Dead Load	742.0		742.0	

Note: “-“ denotes data not considered reliable

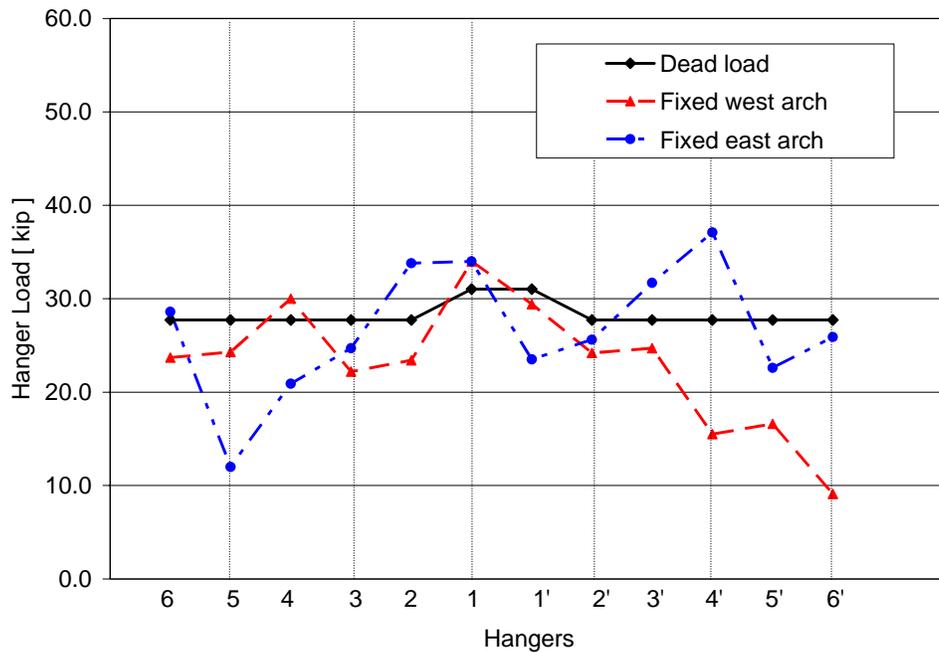


FIGURE 6 Hanger forces for both arches of the 44th Street Bridge after adjustment.

2006 Results; 40th St. Bridge and 44th St. Bridge

The hangers on both the 40th and 44th Street bridges were monitored approximately 1 year after the initial construction to determine if any significant changes or redistribution in the hanger forces had occurred.

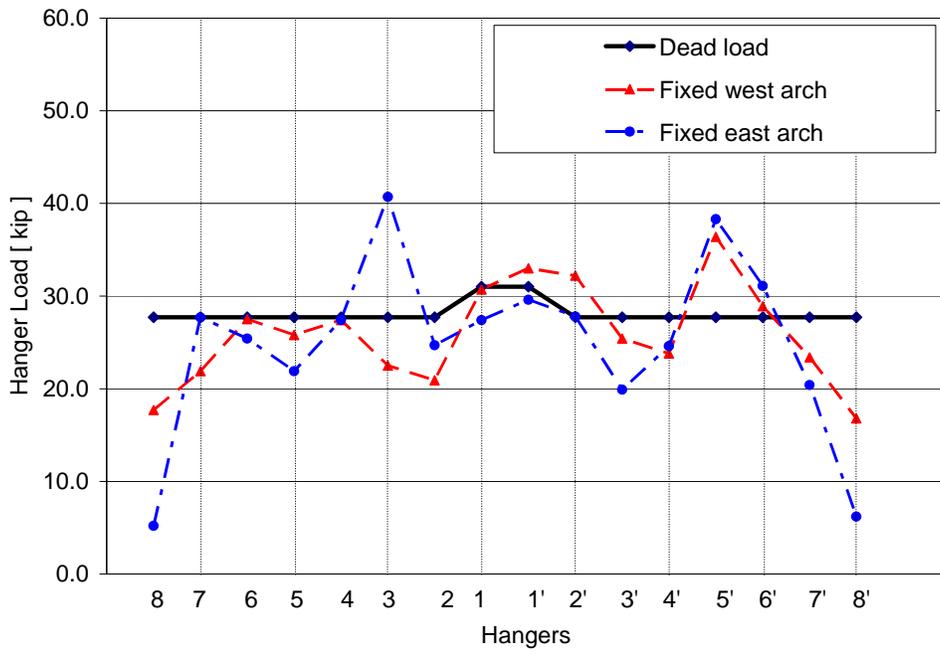
Some force increases were expected due to the increased dead load that was placed after the initial monitoring, primarily due to the addition of a security fences and a deck overlay. The total increase in dead load for the bridges was approximately 96 kips.

Figure 7 shows the hanger forces for the 2005 and 2006 monitoring on the 40th St. Bridge (2005 data have already been discussed in the paper). The calculated dead load forces for each hanger for both years are also shown. Observations noted from comparison of the 2006 data with the 2005 data include the following:

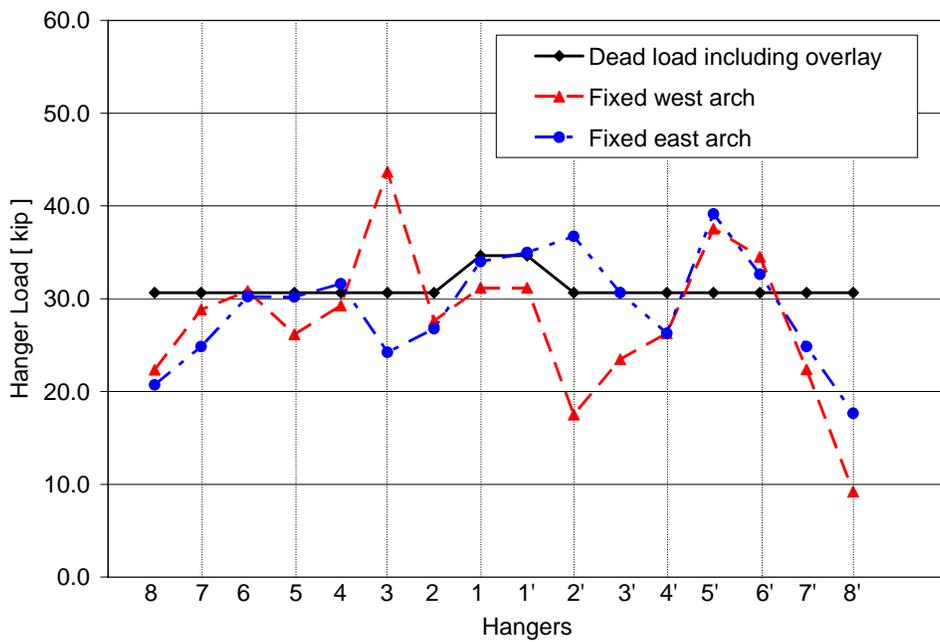
- The total dead load incremented at least 10% from 2005 to 2006 due to the deck overlay and security fence.
- For the 2006 data only a few hanger forces have exceeded the predicted dead load forces.
- The West Arch forces are more unevenly distributed than are the East Arch forces. Hangers 3W and 2'W changed significantly from 2005.

Figure 8 shows the hanger forces for the 2005 and 2006 monitoring on the 44th St. Bridge (2005 data have already been discussed in the paper). The calculated dead load forces for each hanger for both years are also shown. Observations noted from comparison of the 2006 data with the 2005 data include the following:

- The overall distribution of the hanger forces was much more uniform than for the 40th St. Bridge
- There were very moderate changes in the hanger forces during the one year of service.

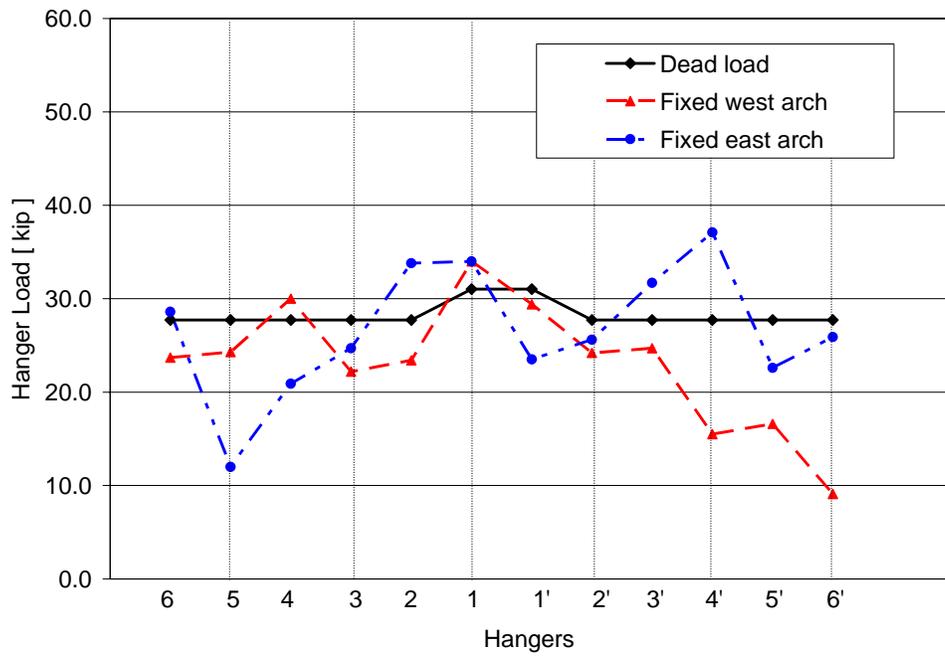


a) 2005 monitoring

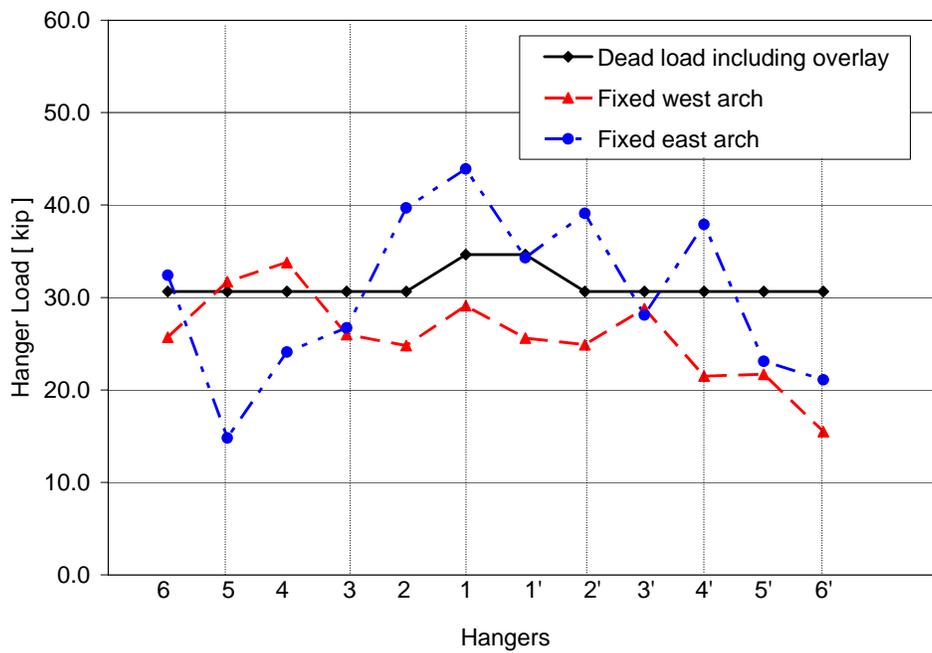


b) 2006 monitoring

FIGURE 7 Hanger forces for both arches of 40th Street Bridge.



a) 2005 monitoring



b) 2006 monitoring

FIGURE 8 Hanger forces for both arches of 44th Street Bridge.

CONCLUSIONS

The accelerated construction of the I-235 pedestrian bridges was completed with only minimal interference to the traveling public which satisfied the primary goal of selecting this specialized structure type. The bridges also provide an aesthetically pleasing “gateway” to the city of Des Moines.

The following conclusions are based on the study results:

- Deck panel cracking caused by heavily loaded hanger rods was significantly reduced during the construction on the 40th and 44th Street bridges.
- The monitoring of the two bridges during construction provided a means for ensuring that hanger loads could be adjusted to a uniform level at both intermediate and final stages of construction. The key to this was ongoing communication and coordination with the contractor, which should be strongly encouraged for future projects.
- The method of hanger measurement has been validated and should be considered a valuable construction monitoring tool for future projects.
- In the year following construction, the changes in hanger forces on the 44th St. Bridge was more moderate than for the 40th St. Bridge. Significant changes in hanger forces occurred at one pair of hangers on the 40th St. Bridge.

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