ABSTRACT

Between 1988 and 1993, the Southeastern Pennsylvania Transportation Authority (SEPTA) undertook a major reconstruction project on the Frankford Elevated rapid-transit line in Philadelphia. Over five and one-half miles of ballasted deck superstructure, originally constructed in the 1920s, was replaced with precast deck units and direct-fixation track. Since the completion of this retrofit, much has been learned about securing a new deck to an existing superstructure.

Economic considerations dictated the use of a haunched precast deck segments in the reconstruction of the original ballasted deck superstructure. The twelve-inch deep haunches are oriented perpendicular to rail traffic and bear on new steel stringers that are continuous through the existing plate girder bents. At each bearing location, precast deck units are anchored by two steel dowels (#5 reinforcing bars) that extend into the haunch. In some locations, the dowels are field-welded to the new stringers while in other locations; dowels and sole plates are cast with the haunches and bolted to the stringers.

By the mid-1990s, SEPTA started to notice problems with the haunches. Concrete at many of the bearing locations was cracking and spalling, posing a threat to both traffic below and to the structure itself. During the summer of 1998, SEPTA approached Gannett Fleming, requesting an in-depth investigation of the deteriorating haunches, including various methods of destructive and nondestructive testing.

The investigation and subsequent analyses concluded that the problems were the result of unintended composite action occurring between the steel stringers and concrete deck haunches. The anchor bars, which were originally installed to keep the slabs from shifting on the stringers, were acting as shear studs, prohibiting the natural tendency for live load slip to occur between the
haunch and stringer. As a result, this live loading from the trains produced large horizontal shearing forces at the steel-concrete interface. Other effects (braking, temperature, centrifugal) contributed to the overall force but were not considered to be the primary cause of damage. The recent introduction of heavier rail cars into the system also magnified the shearing force.

The negative effects were twofold: concrete cracked at some locations and dowels fatigued at other locations. In addition, the deterioration was found to be of a progressive nature, initiating at the expansion ends of continuous units and propagating toward the interior.

The problems encountered on the Frankford Elevated re-emphasize the importance of anticipating and evaluating the effects of connection details, particularly in structure retrofits. Repairs have been recommended which will re-establish the non-composite properties of the system while maintaining lateral restraint of the precast slabs.

INTRODUCTION

The Southeastern Pennsylvania Transportation Authority’s (SEPTA’s) Market Street – Frankford Avenue Elevated Line (MFSE) traverses Philadelphia, Pennsylvania beginning at Frankford Avenue and Bridge Street in the Northeastern section of the City, going through Center City and ending at Market and 69th Streets in the western section of the City (see figure 1). The Northeastern and Western City sections are elevated while the Center City portion of the line is below ground.

This light rail line services 26 stations between its termini in three distinct segments known as:

- The Frankford Elevated which runs above Front Street, Kensington Avenue, and Frankford Avenues from the Spring Garden Station to the Bridge Street Terminal and within the I-95 median from Spring Garden Street to 2nd Street Stations.
- The Market Street Subway which runs under Market Street and the Schuylkill River between 2nd Street and 46th Street Stations
• The Market Street Elevated which runs above Market Street between 46th Street and 69th Street.

Figure 1 - Location Plan

Our segment of concentration, the Frankford Elevated Line or the “El”, is five and one-quarter miles long. It was originally constructed between 1915 and 1922, and began regular service in 1922 between Bridge Street and downtown Philadelphia. Ridership peaked in the 1940’s at about 250,000 riders daily but in the decades following World War II industries relocated out of that section of the City and ridership has fallen to the present 150,000 riders daily. Even with the falling ridership the MFSE remains the backbone of SEPTA’s transit system, with one in five city transit division riders using this Line daily.
Structural problems began to appear in the 1950’s. Between 1952 and 1980, approximately forty-five structural investigations and engineering studies were performed by various consulting firms for both SEPTA and the City of Philadelphia. Major deficiencies were noted in each of these studies. In 1981 SEPTA recognized that major/total reconstruction of this 5-½ mile guideway would need to be performed and hired a consultant to study reconstruction options.

The original structure was supported by single and double column bents consisting of riveted plates for the cross girders and columns. Four steel stringers framed into these cross girders and were continuous over multiple spans. Pin and hanger arrangements provide expansion at one end on a continuous stringer. The stringers supported a ballasted jack-arch deck, which supports two sets of tracks.

The reconstructed structure is comprised of a direct fixation haunched deck in place of the ballasted jack-arch deck and some of the steel superstructure was replaced in-kind (see figure 2). Construction began in 1986 and was completed in 1997.

Figure 2 - Reconstructed Structure
Shortly after completion of this work, the haunches of the precast deck showed signs of deterioration, i.e. spalling and cracking (see figure 3). Over the next several years, under SEPTA’s careful observation, the haunches continued to progressively deteriorate.

![Figure 3 - Spalled Haunch](image)

SEPTA then asked the Valley Forge, Pennsylvania office of Gannett Fleming, Inc. to investigate these haunches, to determine the cause of the deterioration and recommend a repair. The initial area of concentration was in the construction contract section experiencing the most deterioration (Phase 1) and then in the remaining sections (Phase 2). SEPTA’s inspection reports were used, as well as visual field and non-destructive testing investigations. The non-destructive testing was used to determine the presence of hidden delamination in the deck haunches. Concrete cores were used to validate the non-destructive testing.

**INVESTIGATION APPROACH**

To provide these services, the project was performed in two phases. The initial inspection phase and then the supplemental non-destructive testing phase.
Phase I – Field Inspection

In the latter part of 1998, SEPTA contracted Gannett Fleming to inspect a portion of the elevated structure that was experiencing a particularly high frequency of haunch deterioration. The initial cursory inspection included 71 spans with an in-depth view of two continuous units with precast slabs. The focus of the investigation was on the deck slabs, haunches, and their attachment to the steel stringers and cross beams. A structure review was performed on the remaining spans. The intent of this review was to determine whether those spans that were inspected in-depth were representative of the remaining spans.

Analysis

Prior to analysis, Gannett Fleming reviewed available calculations from the design of the Frankford Elevated Reconstruction Project. The input for the superstructures assumed redundant, non-composite properties throughout. Anchor bars were designed for longitudinal and uplift forces only.

Gannett Fleming then performed finite element analyses of the haunches in accordance with the 1983 Philadelphia Transit Design Manual. Live load consisted of a new M4 car at 95-kip crush loading for strength calculations and 90.2-kip peak loading for fatigue calculations. Service load design (with appropriate AASHTO reduction factors) was used in determining existing forces on the structure, in accordance with the original 1991 design criteria. Load factor design was used in designing retrofit schemes.

Alternative Brainstorming

Once the field inspection and analyses were completed, the design team met to discuss/brainstorm possible causes and solutions to the deck haunch cracking/spalling. Each cause was discussed from a theoretical and then a practical viewpoint. Solutions to repair the deck haunch deterioration as well as prevent its recurrence were also discussed as well as advantages and disadvantages for each alternative.
Phase 2

Investigation of Non-destructive Testing Techniques

In late 1999, SEPTA requested that Gannett Fleming examine other portions of the elevated line that were exhibiting similar deterioration. A cursory view of the structure and review of SEPTA inspection records revealed a failure pattern similar to previously inspected units. The haunch/stringer connection detail in this portion was slightly different, but the overall response of the structure was determined to be typical.

Once the typical visible deterioration pattern was established, attention was focused on unseen delamination and a nondestructive testing program was initiated on a sampling of haunches. The goal was to determine the extent to which visually sound haunches had unseen distress. Various nondestructive testing techniques were investigated and the impact-echo method was found to be most appropriate for the site. This method uses a point impact to generate a stress pulse, which travels through the concrete and reflects back to a receiving transducer on the surface. The pulses reflect back at different intervals, depending on whether a crack is present (see figure 4).
Wiss, Janney Elstner Associates, Inc. (WJE) of Northbrook, Illinois was contracted to perform the testing and technique comparison. Approximately 300 test locations were identified to provide an effective sampling of haunch conditions throughout the structure.

**Haunch Removals**

After NDT was complete, total concrete removal of four haunches was conducted to determine the condition of the steel reinforcing dowels, because in Phase 1 destructive testing uncovered fatigue cracks in the dowel bars. The procedure required removal of the bottom six inches of the haunch. After examining the vertical steel dowels and spacer plates/bars, removed haunches were replaced by injecting an epoxy into a preplaced aggregate.

**Analysis, Recommendations and Reporting**

Once the testing was complete, the results were analyzed and conclusions were drawn regarding the nature of the haunch deterioration. Based on these conclusions, Gannett Fleming evaluated the anticipated effectiveness of SEPTA’s proposed repair scheme and other possible alternatives. A report was then submitted discussing findings and recommendations.

**FIELD INVESTIGATON FINDINGS**

**Phase 1**

The initial cursory inspection included 71 spans at the south end of the line, with an in depth inspection of some of the precast units with a high frequency of deficiencies. These units were five to seven spans long and were comprised of precast concrete deck panels anchored to four longitudinal steel stringers by pairs of anchor bars welded to the stringer’s top flange. The spans were continuous at interior bents and expand through pin-and-hanger mechanisms at end bents. Precast panels were originally fabricated with preformed vertical holes, set in place over the welded anchor rods, and grouted in position.
In each unit, concrete haunches were found to be severely delaminated or spalled in the end spans near the expansion devices (see figure 5). In nearly every case, the deterioration was on the face of the haunch opposite the expansion device and was more prominent over fascia stringers than interior stringers (see figure 6). Moving away from the pin-and-hangers, cracks generally decreased in size and frequency. Haunches near the fixed-end bents were usually sound.

![Figure 5 - Phase I Haunch](image)

Cracks typically initiated at the base of the haunch, directly over the stringer, and propagated upward. These cracks could be seen opening and closing during passage of live load. Many of the cracks had progressed completely through to the top of the haunch, causing a large crescent-shaped piece to spall off and fall to the stringer top flange or to the roadway below. At spalled locations, the haunch bearing was typically reduced by 50% or more. Expansion mechanisms and deck joints were examined closely and found to be moving freely (see figure 7). An inspection of the track determined that placement of high and low-restraint track fasteners were appropriate for the expected structure expansion and contraction movements. As a result, "frozen" bearings and rail temperature effects were not considered to be major factors causing
the haunch damage. Haunch behavior was also examined during passage of live load. Differential movement of up to 1/8 inch in each direction was noted between the haunch/stringer interface at both cracked and uncracked locations. This type of movement was noted in both interior and expansion spans, but was more pronounced in the expansion spans. GF followed up the initial inspection by removing the lower portion of four haunches to observe the condition of the steel anchor bars and their attachment to the stringer top flange. One haunch, noted as being in good visual condition, was chosen as a control for and three others chosen which exhibited cracks or differential movement. The anchor bars were cracked at many locations, including the
visually "undamaged" control haunch which had bars cracked approximately half way through. Dye penetrant testing was used to confirm the presence of cracks. A few of the anchor bars were sawed clear to get a view of the crack surface (see figure 8). The crack pattern on the broken bars was indicative of fatigue failure, not a sudden shear failure (see Figure 9). The crack was typically in the bar, not the weld, and in no case did the crack propagate into the stringer flange.

Figure 7 - Expansion Mechanism

Figure 8 - Bent Anchor Bar
A correlation was drawn between those haunches with noted differential movement at the interface and those found to have fatigued bars. These field results were used in the analysis to develop an actual fatigue threshold for the anchor bars.

**Phase 2**

The haunch/stringer connection detail in this portion was slightly different (see figure 10), but the overall response of the structure was determined to be typical. These haunches were attached to the stringers via steel plates and bolts, some of which had slotted holes to allow longitudinal movement; however, discussions with SEPTA revealed that many of the expansion bolts were mistakenly tightened during construction. This secured the haunch to the stringer and thereby created a rigid (non-slip) connection similar to that encountered in the earlier section (phase 1). These bolts were loosened once the haunches began to exhibit deterioration.

Appropriate nondestructive testing methodology and representative test sites were chosen, then testing was performed during overnight outages to minimize live load distortion and disruption of roadway traffic below. Of these test sites, nearly 17 percent exhibited hidden delamination or cracking. Concrete coring was used to verify the NDT results and confirm the design strength of
the precast concrete. Upon removal of select haunches, the dowel bars were found to be in good condition with no signs of fatigue as seen in the earlier phase 1 investigation.

**ANALYSIS**

The inspections revealed two basic deficiencies in the precast concrete units:

- **Case 1:** Haunches near the expansion ends have visibly cracked concrete and/or cracked anchor bars. Seventeen percent of other haunches have hidden internal cracks.
- **Case 2:** Other haunches away from the expansion ends have concrete in good condition but still have broken or fatiguing anchor bars.

The analysis focused on determining the causes of these deficiencies and was divided into two basic mechanisms: strength and fatigue.

**Strength Analyses**

The haunches were first analyzed for direct horizontal loads only (impact, braking, centrifugal, temperature), using the AASHTO shear friction design method. The two 5/8 inch diameter
anchor bars were found to have adequate capacity to transfer those horizontal forces to the stringers. The values below show the concrete to be adequate in a shear or bearing type failure for these loads:

<table>
<thead>
<tr>
<th>Case</th>
<th>Force (k)</th>
<th>Capacity (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Friction</td>
<td>10.2</td>
<td>10.5</td>
</tr>
<tr>
<td>Concrete Shear</td>
<td>10.2</td>
<td>19.0</td>
</tr>
<tr>
<td>Bearing</td>
<td>21.9</td>
<td>103.5</td>
</tr>
</tbody>
</table>

The analysis was then refined to include the possibility of unintended composite action occurring between the stringers and the precast slabs. This scenario seemed likely, particularly since the connection details provide no mechanism for anticipated non-composite slip between the haunch and steel. Two-dimensional STAAD models were developed. Pin-and-hanger supports were modeled as rollers, interior bent supports as fixed, and shear connections as pins. Composite action was assumed throughout, including negative moment regions where massive cast-in-place end pours fasten the deck rigidly to the bent crossbeams.

The following criteria were used in strength analysis of the concrete haunches and steel anchors:

- One 6-car M4 (95k crush load) train on each track (wheel load distribution factor WLDF = 1.0)
- Centrifugal force (CF) included, where appropriate, as additional wheel load on outermost track
- Absolute max(+)/min(-) shear selected for one-time loading
- 5000 psi concrete and Grade 60 anchors rods, per as-built general notes
- Temperature force (13.5k/haunch) due to differential expansion between concrete slab and steel stringers

STAAD runs, which included the deck in the model, confirmed the general damage pattern noted in the field. As shown in figure 11, the horizontal shear force in expansion spans ranged between
30 kips and 65 kips per haunch, depending on location within the span. In locations nearest the expansion ends, these forces were high enough to exceed the ultimate shear capacity of the concrete (40.5k/haunch); hence, the haunches cracked on the inside face (see figure 5), opposite the expansion end of the span. As noted during the inspection, the frequency of concrete failures was greater at fascia stringers, which saw higher live load forces due to CF and higher distribution factors, thereby producing higher horizontal shear.

**Fatigue Analyses**

Although most of the absolute horizontal shearing forces (above) did not exceed the shear strength capacity of the anchors (45.7k/haunch), the shear stress ranges due to composite action greatly exceeded the anchor capacity in fatigue (acting as a shear stud). From inspection observations of five fatigue cycles per train and study of the structure's history, it was determined that each haunch/stringer connection in phase 1 had seen approximately 2 million cycles in the 8 years since completion of reconstruction.

![Figure 11 - Haunch Concrete Capacity](image-url)
Haunch forces range from 28.1 kips to 53.5 kips per haunch, using a reduced live loading to represent a fatigue vehicle. From AASHTO criteria for shear studs, the theoretical capacity of the two anchors in fatigue is 4.3 kips for over 2,000,000 cycles; This value was obviously too conservative, since field observations showed many of the bars to be still intact. Field observations of broken/unbroken anchors were used to bracket the actual fatigue threshold. The actual estimated fatigue capacity is approximately 26 kips/haunch, which was still below the observed range (see figure 12). For this reason, it was estimated that bars will continue to fatigue in a progressive fashion as cycles increase.

![Figure 12 - Anchor Bar Fatigue Capacity](image)

**Retrofit Analyses**

The STAAD analyses indicated that it is not advantageous to retrofit portions of spans. For the non-composite alternatives, as haunches are freed (i.e. concrete cracks or bars fatigue), the shearing forces at adjacent "fixed" haunches and at the fixed end bent connection rise significantly (sometimes > 100%). In some cases, these forces could be high enough to cause additional spalling in the concrete. However, if all haunches are freed simultaneously, live load
shearing forces on the haunches are eliminated and taken by the massive concrete end blocks, which are fixed, to the bents.

**CONCLUSION**

Horizontal shear force resulting from composite action is the primary force causing damage to the haunches (see figure 13). Centrifugal force, impact, longitudinal force, wind, and temperature effects are minor comparatively. The horizontal force produces two distinctly different failure mechanisms in the haunches, concrete shear (occurring almost immediately) and anchor bar fatigue (occurring over time). Concrete shear occurred first. The ultimate shear strength of the haunch concrete is less than the ultimate strength of the welded anchor rods; hence, at the free ends of the units where forces are highest, the haunches spalled almost immediately while the anchors remained firm. Inspection findings confirm this scenario with the majority of spalls occurring near the expansion devices on the opposite face. Since the spalls are a direct result of live loading, we would expect a greater frequency of failures where the load is highest; again, inspection findings confirm this hypothesis in units where centrifugal force is concentrated on the outside fascia stringer. The deficiency layout sheets clearly show a greater number of problems in those areas (see figure 6).

![Figure 13 - Beam/Deck Non-Composite Response](image)

*Figure 13 - Beam/Deck Non-Composite Response*
At other locations, the composite forces did not exceed the concrete shear strength but did exceed the fatigue capacity of the anchors. Anchors fatigued first in the haunches with the highest horizontal forces and continue to fatigue in a progressive manner throughout the other haunches. The introduction of a heavier car in the early 1990’s accelerated this progression. Inspection findings also verify these conclusions; haunches with perfectly intact concrete were found to have fatiguing anchors while haunches that exhibited slip between surfaces were found to have broken (fatigued) anchors.

In the Phase 2 study area the precast concrete deck panels are attached to the stringers via plates and bolts. These plates are slotted at the ends of the spans and use standard sized holes in the center portion of the spans. These details recognize the tendency for the deck and beam to act compositely and attempted to limit these shear forces. Unfortunately, in most of these construction sections the bolts at the ends of the spans were tightened, forcing the entire deck and beam to act compositely and consequently, initiated haunch damage. Once this damage was found, the bolts at the slotted holes were loosened. However, the number of damaged haunches still increased probably because some of the haunches were delaminated and the delaminations progressed into a visible crack or spall.

Analyses predict that the damage to the haunches will continue until all of the haunches have been freed. Loosening the bolts in the slotted holes has allowed the structure to act as intended; however, it appears that all of the haunches will experience overloads under the newest live loading and will eventually fail.

The hidden delaminations found appear to be the initiation of haunch deterioration. Haunch stresses build until the concrete initially delaminates, then cracks, and finally spalls. Once a haunch has spalled it has relieved its composite stress. These stresses then redistribute and this process begins on the next haunch towards center of the span. This progressive failure will continue until all of the haunches have failed/spalled.
REPAIR ALTERNATIVES AND RECOMMENDATIONS

Repair schemes can be broken down into two basic alternatives: composite and non-composite retrofits. The intent of a composite retrofit is to provide a continuous, rigid connection between the deck and stringers, thereby eliminating any slip potential and increasing the live load carrying capacity. Non-composite retrofits seek to completely free the haunches from the effects of live load horizontal shear by allowing controlled slip to occur.

Phase 1

Details for each scheme in this Phase are shown in figures 14, 15 and 16. The non-composite alternative is further subdivided into two possible slip mechanisms: one utilizes a neoprene pad and one uses sliding plates. Advantages and disadvantages of each scheme were analyzed for the characteristics listed hereafter to help determine the relative attractiveness of each alternative:

- Up Front Cost - Initial cost of repairs
- Lifetime Cost - Initial cost + ongoing maintenance costs or replacements
- Constructability - Relative complexity of the repair; can repair be staged?
- Performance - Estimated life of repair; does repair act as designed?
- Nondestructive - Impact of the repair to existing load-carrying elements
- Inspectability - Can the repair be easily inspected for future performance?
- Quality Control - Relative ease of monitoring quality of materials/field operations
- Service Interruption - Are outages required to complete the repair?

Programming both composite and non-composite retrofits into stages was investigated. The stages were broken down into priority (1 year), short term (3 years), and long term (10 years) repairs. These stages were:

- Priority repairs: restores bearing to severely deteriorated haunches (expansion spans).
- Short term repairs: make composite or non-composite those haunches, which have little
remaining fatigue life

- Long term repairs: make composite or non-composite those haunches, which are anticipated to be fatigued within the next 2 million cycles

Figure 14 - Composite Repair Alternative

It was found that the non-composite alternative that included the neoprene pad was least expensive (particularly for short-term repairs), easiest to construct, least disruptive, and permits most parts to be fabricated in the shop. This system was also the most flexible, allowing the deck and stringers to function as truly non-composite entities while still providing lateral resistance against braking, centrifugal, impact, and temperature forces.

Figure 15 - Non-Composite Repair w/Neoprene Pad
Only priority and short term repairs were recommended since the secondary STAAD analyses, which include the deck in the model, indicate that it is not advantageous to retrofit portions of spans. For the non-composite alternatives, as haunches are freed, the shearing forces at adjacent haunches and at the fixed end bent connection rise significantly (sometimes > 100%).

![Figure 16 - Non-Composite Repair w/Sliding Plate](image)

In some cases, these forces could be high enough to cause spalling in the concrete. However, if all haunches are freed simultaneously, live load shearing forces are eliminated; hence, repairs should be performed on the entire span. As a result, this scenario precludes use of long term repairs, which were applicable only to portions of two spans. All repairs should therefore be categorized as priority or short term.

**Phase 2**

Repair schemes were again broken down into the same two basic alternatives as in Phase 1. However, loosening the existing bolts connecting the haunch bearing plate to the stringer, provides controlled slippage. Also, reducing the bolt size to ¾ inch at the plates with slotted holes and to ½ inch at the plates with normal sized holes provides the movement required to theoretically eliminate damage to the haunches. The standard size holes were recommended to be reamed to 1 ¼ inches and the slotted holes verified to be at least 1 ½ inches long in locations greater than 60 feet from the midpoint of a unit. Damaged haunches were recommended for repair. A composite alternative was not considered since it was obvious that its cost was much greater than the non-composite alternative.