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Joints: the weak link in bridge structures and lifecycles

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Abstract. The condition of the vehicular bridge network in New York City, as represented by ratings obtained during biennial inspections is reviewed over a period of three decades. Concurrently, the bridges comprising the network are considered as networks of structural elements whose condition defines the overall bridge condition according to New York State assumptions. A knowledge-based matrix of assessments is used in order to determine each element's vulnerability and impact within the network of an individual structure and the network of City bridges. In both networks expansion deck joints emerge as the weak link. Typical joint failures are illustrated. Bridge management options for maintenance, preservation, rehabilitation and replacement are examined in the context of joint performance.

Keywords: bridge; design; deterioration; joint; life-cycle; maintenance; management; rehabilitation

1. Introduction

Processes and products commonly fail at discontinuities. Engineering structures most frequently fail at joints. This is particularly true of bridges where joints function under dynamic conditions in aggressive environments. Hence, the statement that "the only good joint is no joint" is claimed by numerous contributing authors. That thinking is pursued in two directions. On bridges of up to several relatively shorter spans numerous publications, including Burke (2009) and Azizinamini *et al*. (2013) recommend jointless bridges and integral abutments. The large cyclic displacements of long-span bridges are accommodated by special expansion devices, such as the modular joints, designed and manufactured to detailed specifications including installation, maintenance and replacement. Between these two extremes, both options co-exist in the vast population of multi-span bridges serving vehicular traffic for many decades. The joints of such bridges can be limited to fewer locations and designed for larger movement, or located at every second span and built for smaller displacements and greater impact.

The performance of expansion joints has been assessed quantitatively on project levels and qualitatively on network levels. Syntheses, such as Burke (1989) and Purvis (2003) have identified typical modes of failure and expected lifecycles of specific joint types. Element- and span-specific biennial inspections reports generate qualitative condition ratings allowing for comparisons between bridge elements, structures and entire networks. Attempts to correlate the findings point to deck joints as a particularly vulnerable link in both the product of design and in the process of

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management. Consequently, their deficiencies must be addressed on all stages and levels of the bridge life-cycles. On a network level, the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Specifications amplify the design live load for joints by an impact factor of 75%. On project levels, joint malfunctions persist in construction, maintenance and operation. In order to fully address the problem therefore, the product of bridge design must be examined as a network of structural elements, whereas the process of network management must be reviewed as a product of the most pressing needs for urgent action.

2. The vehicular bridge network

In 2013 New York City is responsible for 789 vehicular and pedestrian bridges with approximately 5,000 spans and a total deck area of 1.5 million m^2. Among them are the Brooklyn (1883), Williamsburg (1903), Manhattan (1909) and Queensborough – Ed Koch (1909) and 25 moveable bridges (swing, lift, bascule and retractile), as well as many multi-span viaducts. The bridges are inspected at least biennially according to New York State standards. The condition of all elements in all spans is rated from 7 (new) to 1 (failed). The overall bridge condition rating R is calculated according to the formula of Eq. (1). It is a weighted average of the lowest ratings of 13 structural elements. The elements i and their respective weights w_i are shown in Table 1.

$$R = \sum_{i=1}^{13} w_i R_i \tag{1}$$

where:

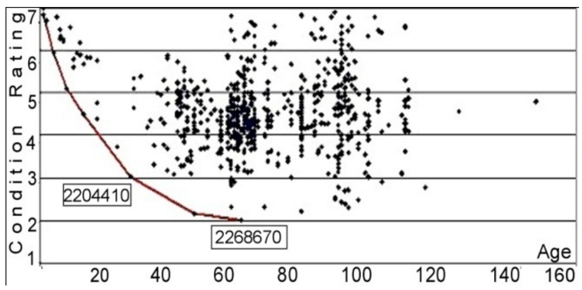
R_i - lowest condition rating of element i observed on the bridge during a regular inspection (not necessarily in the same span)

w_i - weight assigned to element i.

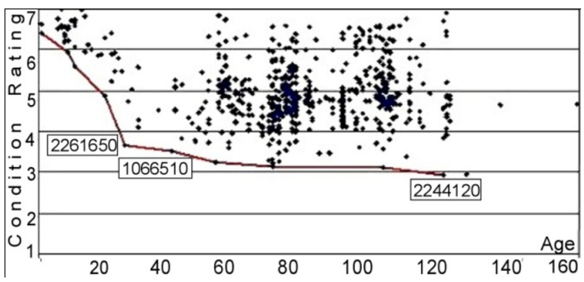
The condition ratings obtained by Eq. (1) represent mostly deterministic and qualitative visual assessments. After more than 30 years of accrual, they appear suitable for statistical analysis. Such analysis however treats the data as homogenous, thus overlooking the following significant information. The work invested in each structure varies widely depending on its age, size, and function. Condition ratings $R > 4.5$ at bridges older than 50 years invariably imply substantial investments in rehabilitation. To illustrate the point, the condition ratings of the New York City bridges for 1990 and 2012 are plotted with respect to their "year built" in Figs. 1(a) and 1(b). Based on similar data, Yanev and Chen (1993) found the bridge condition ratings to be normally distributed, with a mean of roughly 4.5. An average deterioration rate obtained from either set of data points in Fig. 1 would never drop below the rating of 3 (not functioning as designed). Moreover, a cluster of bridges aged between 80 and 100 years are in very good condition. In contrast, the data points representing the lowest rated bridges for any age (denoted by the respective Bridge Identification Numbers in Fig. 1) have risen significantly over the examined period. This is easily understood when taking into account that approximately $US 4 billion have been spent over the last 30 years at the four East River crossings alone, even though they remain "built" in 1883, 1903, 1909, and 1909, respectively. New York City DOT has spent roughly $US 500 million annually on captial bridge projects for over a decade. Thus Figs. 1(a) and 1(b) illustrate the bridge network improvement as a result of capital reconstruction.

Table 1 Elements and weights in the bridge condition formula, NYS DOT

i	1	2	3	4	5	6	7	8	9	10	11	12	13	∑
Element	Bearings, anchor bolts, pads	Backwalls	Abutments	Wingwalls	Seats	Primary member	Secondary member	Curb	Sidewalk	Deck	Wearing surface	Pier	Joints	
w_i	6	5	8	5	6	10	5	1	2	8	4	8	4	72



(a) 1990



(b) 2012

Fig. 1 Condition rating versus age for the New York City bridges in 1990 and 2012

During the 1990s the annual expenditures for capital reconstruction & component rehabilitation peaked at $US 600 million. Yanev (2007) demonstrated that the condition rating improvements resulting from these expenditures are in equilibrium with the fastest, rather than the average rate of deterioration. The steepest line of decline consistently approaches the rating of 3 in 30 years. Over the period illustrated in Figs. 1(a) and 1(b) the average overall bridge condition did not change very significantly. In contrast, the lowest condition ratings no longer fall below 3 in 2012.

The network level equilibrium between serviceable bridge conditions and related expenditures is currently maintained at an annual cost of about $US 400/m^2. Having eliminated the rigid constraint of service failures presented by poor bridges through reconstruction / rehabilitation, management can further optimize direct and user life-cycle costs by maintenance / preservation.

3. The bridge element network

According to Section 2, the annual reconstruction / rehabilitation costs of a large network are governed by the bridges rated below the acceptable condition level. Similarly, if individual structures are modeled as networks of elements, their annual maintenance / preservation costs under budget constraints should depend on the elements with the fastest rate of deterioration and the greatest impact. With condition ratings for all bridge elements in all spans, the New York State bridge database supports such modeling.

Agrawal and Kawaguchi (2009) obtained element deterioration rates for New York State bridges by Markov and Weibull models, ultimately recommending the latter. The resulting life-cycles are consistently around 75 years. As for bridges, so for their elements, the deterioration rates obtained from large databanks by statistical methods systematically obscure the worst and hence, most important cases. For the vehicular bridges in California, rated by the AASHTO Guide for Commonly Recognized Structural Elements, Thompson and Johnson (2005) concluded that the Markovian model alone is "inadequate … to estimate transition probabilities from a historical inspections data set alone. Important information is clearly missing. The authors believe that this missing information is maintenance activity."

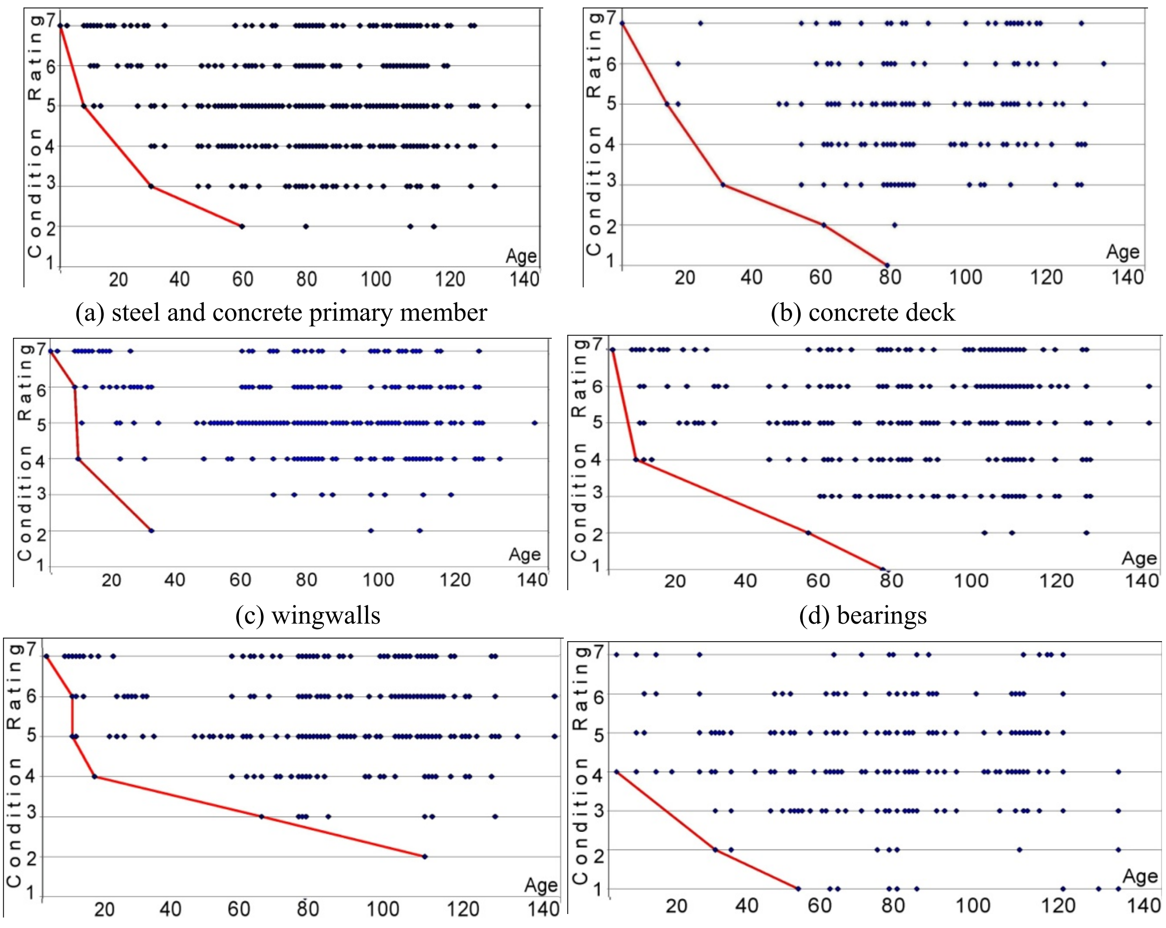
Fig. 2 shows the condition ratings of certain structural elements included in Eq. (1) with respect to their age for the New York City vehicular bridges in 2012. Once again, rather than deriving averages, lines are drawn through the worst condition ratings obtained at any age. It can be observed that the elimination of the worst bridge conditions, shown in Fig. 1(b) is not matched by a corresponding improvement in the worst element conditions in Fig. 2 for that year. Consequently, whereas Figs. 1(a) and 1(b) illustrate overall bridge condition improvements obtained by reconstruction / rehabilitation, Figs. 2(a) - 2(f) do not show element condition improvement that might result from maintenance / preservation. Whether spontaneous or deliberate, such a management choice merits examination.

Yanev, Testa and Garvin (2003) and Yanev and Richards (2011) optimized network lifecycle expenditures by correlating the levels of maintenance task performance and the corresponding element life extensions. The 15 maintenance tasks, recommended by Bieniek *et al*. (1989, 1999) were assumed to influence the condition ratings of the 13 structural elements of Eq. (1) according to deterministically selected importance factors. The shortest known lifecycles of the bridge elements, such as those in Figs. 2(a)- 2(f) are assumed to occur at no maintenance.

The described method treats structural elements as independent. Bridge design similarly considers bridge structures as sets of discrete elements, to be dimensioned for prescribed loads. This potential discontinuity in the design process must be deliberately eliminated before it translates into a discontinuity of the final product. The design of details transferring loads between elements came under intense scrutiny in 2007 after the gusset plate failures at the deck truss bridge carrying Rte. I-35 over the Mississippi River.

An alternative view is proposed in order to better reveal structural vulnerabilities that might be suitable targets for bridge maintenance / preservation, before reconstruction / rehabilitation become inevitable. In the absence of qualitative / quantitative data, bridge managers have to rely on the engineering knowledge gained from inspection, maintenance, and particularly from the evidence of structural element failures. The set of values in Table 2 are one example. They are assembled as follows. The bridge structure is considered as a network comprised of the 13 elements in Table 1 and Eq. (1). The impact of each element on the others is rated from 0 to 1 by increments of 0.1 along the rows. As a result, the sum of each row in column 14 represents the estimated impact of the respective element within the structural network. Once the matrix is completed in this manner, the sums of each column shown in row 14 would conversely represent the estimated vulnerability of each element to the rest. Alternatively, one could estimate vulnerabilities along the columns, and the impact would obtain along the rows. Elements are assigned impact / vulnerability of 1 with respect to themselves, accounting for the values of 1 along the main diagonal of the matrix. The values are deterministic, as were the weights w_i in Table 1, and reflect knowledge of structural behavior and the worst case trends exhibited in Figs. 1 and 2.

The “impact” and “vulnerability” indexes in Table 2, column and row 14, respectively, are comparable to the values of w_i in Table 1, with some significant differences. The most vulnerable elements of Table 2 are primary members, bearings, anchor bolts & pads, and decks. Leading in impact are joints, decks and wearing surface. Primary members, joints, and decks, followed by bearings obtain highest cumulative scores, suggesting most critical combinations of impact and vulnerabiliy. Of these elements, joints have by far the shortest useful life, as shown in Fig. 2(f). Thus they can be regarded as the weak link in the bridge structural chain, best suited for early maintenance / preserevation actions. That distinction is shared by scuppers and paint, which are not included in Eq. (1). Their inclusion in the equation and in Table 2 would improve the model of the structural vulnerability to early deterioration.



(a) steel and concrete primary member
(b) concrete deck
(c) wingwalls
(d) bearings
(e) backwalls
(f) joints

Fig. 2 Condition ratings versus age for critical bridge elements in 2012

(a) armor joint, concrete

(b) monodeck, torn angle

(c) protruding seal

(d) missing seal

(e) concrete header

(f) butt joint, elastomeric header

(g) poured in place seal

(h) cushion joint

(i) plug joint

Fig. 3 Typical malfunctions of deck joints

Table 2 A sample "correlation" between the conditions of the critical bridge elements

i		1	2	3	4	5	6	7	8	9	10	11	12	13	14
	Element	Bearings, anchor bolts, pads	Backwalls	Abutments	Wingwalls	Seats	Primary members	Secondary members	Curb	Sidewalk	Deck	Wearing surface	Piers	Joints	∑ (Impact)
1	Bearings	1	0.5	0.5	0.2	1	1	0.3	0	0	0.3	0.1	1	0.8	6.7
2	Backwall	0.7	1	0.3	0.3	0.4	0.4	0	0	0	0	0	0	0	3.1
3	Abutments	0.7	1	1	1	0.8	0.4	0.2	0	0.4	0.1	0.2	0	0.2	6
4	Wingwall	0	0	1	1	0	0	0	0	0.3	0	0	0	0	2.3
5	Seats	1	0.3	0.4	0.1	1	1	0.2	0	0	0.3	0	1	1	6.3
6	Primary members	1	0.5	0.5	0	1	1	1	0	0	0.9	0	0.8	0.8	7.5
7	Secondary members	0.2	0	0	0	0.2	0.8	1	0	0	0	0	0.2	0	2.4
8	Curbs	0	0	0	0	0	0.2	0	1	1	0.8	0.9	0	0.2	4.1
9	Sidewalks	0	0	0	0	0	0.2	0.1	1	1	0.8	0.5	0.2	0.1	3.9
10	Deck	0.8	0.2	0.2	0	0.3	0.9	0.5	1	1	1	1	0.7	1	8.6
11	Wearing surface	0.7	0.2	0.4	0	0.7	0.8	0.4	1	0.4	1	1	0	1	7.6
12	Piers	1	0	0	0	1	1	0	0	0	1	0	1	0.7	5.7
13	Joints	1	0.5	0.5	0	0.8	1	1	0.6	0	1	1	0.7	1	9.1
14	∑ Vulnerability	8.1	4.2	4.8	2.6	7.2	8.7	4.7	4.6	4.1	7.2	4.7	5.6	6.8	73.3
15	∑ Vuln. + ∑ Impact	14.8	7.3	10.8	4.9	13.5	16.2	7.1	8.7	8.0	15.8	12.3	11.3	15.9	

4. The weak link: deck joints

In 2013, 677 of the 789 bridges on the New York City inventory were vehicular, with approximately 4,000 spans. The total number of joints exceeded 3,000, of which 1409 were expansion ones. Figs. 3(a)-3(i) illustrate typical joints and their malfunctions. Prevalent are the steel armored joints with rubber seals shown in Figs. 3(a)-3(e). Rarer are the "cushion" joints, as in Fig. 3(h). The butt joints with or without steel edges, shown in Figs. 3(f)-3(g) are used in decks with minimal thermal movements. Elastomeric concrete headers, as in Fig. 3(f) are replacing the regular concrete ones. Polymer poured-in-place sealants are used in decks without armor, as in Fig. 3(g). Proprietary inflatable seals are used for joints accommodating slightly larger displacements than the butt joints. The extreme traffic hazards of failing armor and cushion joints are temporarily mitigated by plug joints, as in Fig. 3(i), by paving over with asphalt or, ultimately by steel plates, as in Fig. 4(a). The practices, malfunctions and failures reported by Purvis (2003) pertain in all of the represented cases.

Typical malfunctions observed by NYC DOT Bridge Inspections include the following:
Compression seals: debris accumulation, ruptured, protruding, missing.

Poured-in-place seals (butt joints): drying out, debonding;
Steel armor: loose, broken, protruding;
Cushion joint: broken bolts, loose, missing plates;
Concrete header: cracked, sagging, broken, exposed re-bars;
Elastomeric header: poor adhesion of sealant;
Concrete decks: broken corners;
Asphalt pavement: cracked, sinking;
Plug joints: cracked, eroded strip, bulging, sinking;

Modular & finger joints are not included herein due to their special functions and properties. They are particularly vulnerable to misalignment, impact and debris accumulation. Their impact on the serviceability of large bridges is critical.

The impact of joint failures on the vulnerable structural elements is irreversible. Figs. 4(a)-4(d) show damage to decks, bearings, pedestals, fascia and entire piers under expansion joints. The rest of the bridge shown in Fig. 4(d) appears in good condition.

(a) steel plate replacing a failed joint (b) spalling fascia

(c) bearings, pedestals under expansion joint (d) pier rehabilitation at expansion joint

Fig. 4 Structural consequences of expansion joint malfunctions

5. Conclusions

Products and processes fail at discontinuities arising between their components and their stages, respectively. The life-cycle performance of expansion joints identifies them as discontinuities both in the designed product and in the management process. Designed and built for an unrealistic type of service and maintenance, they are managed with unrealistic performance expectations. Their replacement is often the only available remedy, amounting to rehabilitation, because of its complexity. As each stage of the process relies on the next one to improve the performance of the product, the useful life of joints is effectively minimized. Thus steps towards improving the performance of the product must be taken at all stages of the process.

- Design

Fu *et al*. (2003, p. 15) conclude that decks are primarily designed and reinforced for flexure, but fail in "shear fatigue". Fatigue design is well advanced for steel, but not for the other involved materials, including concrete, elastomeric concrete, polymer sealants, and so on. AASHTO LRFD has emphasized the vulnerability of joints to fatigue and fracture by increasing their live load impact factor from 33% to 75%. The increase however, reflects primarily the performance of modular joints and is likely to be used in their design. The higher impact factor does not extend to decks. There is abundant evidence that if the steel armor does not fail, it destroys the concrete header and if the header holds, it breaks away from the deck. The following questions arise:

What are the appropriate dynamic loads acting on joints?

What materials and dimensions can resist these loads over an acceptable number of cycles?

While these questions remain unresolved, most expansion joints are not designed for a specific performance, but adopted from a "state-of-the-art" inventory of available supplies.

- Construction / Rehabilitation

The steel and concrete joint elements and compression seals are usually installed according to design specifications for the product, which is typically limited to the joint itself. Minor structural misalignments can impose unsustainable loads on joints. The poured in place seals are sensitive to ambient conditions and material preparation. As a result, their performance is highly uneven. Adhesion to elastomeric and regular concrete headers is poor. There are no warrantees covering their expected useful life under AASHTO traffic loads.

- Maintenance / Preservation

Compression and poured in place seals are not regularly cleaned and replaced. Loose steel angles are not bolted until they break or break the concrete header. Bearings, pedestals and troughs under expansion joints are not cleaned. Plug joints can be installed as emergency replacement of cushion joints with an expected life of 2 - 4 years, depending on the traffic volume and the demand for structural movement.

All joint-related maintenance tasks, including the inspections require extensive traffic closures and incur high direct and user costs. Traffic and water under the bridge further constrain remedial work. As a result, lapses in joint maintenance typically accelerate reconstruction, as illustrated in Figs. 4(a)-4(d).

Carol (2013) has investigated the dynamic performance of polymer concrete as a partial repair for failed joint headers. The findings underscore the importance of taking into account dynamic loads and avoiding material discontinuities, such as the ones arising between steel and concrete in armor joints.

The elements comprising the bridge condition formula in Eq. (1) and enumerated in Tables 1 and 2 have been chosen more than 35 years ago in order to prioritize rehabilitation / reconstruction. The expansion joint deficiencies described thus far and the potential countermeasures remain below the scope of reconstruction, but exceed “routine” maintenance. The Bridge Preservation Guide (FHWA, 2008) addresses that gap in the management options. It contains a set of activities designed to arrest at relatively low cost the deterioration of elements with high impact before the critically important decks, primary members and piers are affected. Condition rating trends, such as the ones shown in Figs. 2 suggest that scuppers would be a suitable target for bridge preservation, along with expansion joins. Painting is already treated as rehabilitation.

- Condition Assessment

The present conclusions are drawn from qualitative visual assessments of structural conditions and loosely related expenditures quantified in money. Since there is no uniquely defined correlation between the two, the costs of maintenance / preservation activities are known, but their benefits in terms of life extension are at best assumed. Over the period reflected in Fig. 1, structural condition assessments emphasized the elimination of imminent hazards, ultimately achieved by reconstruction / rehabilitation. As the needs & benefits related to maintenance / preservation gain in importance, so does the demand for their quantification. That demand has yet to be met, with the help of the newly developed capabilities for non-destructive testing and evaluation (NDT&E). Current long-term bridge health monitoring initiatives have concentrated on the performance of bridge decks. There are also advances in the monitoring of seismic performance and scour. The performance of expansion joints under traffic is another monitoring target which would advance bridge design.

So far, joint malfunctions are fully addressed when their consequences, such as the failure of decks, bearings and pedestals become manifest. In the meanwhile, vehicular traffic is growing in volume and gaining in weight. Joints and their “headers” regularly break under cyclic or unique impact. The “impact” and “vulnerability” estimates in Table 2 represent an assumed structural response to assumed traffic loads. Without quantifiable measurements, it would be hard to base important management decisions on the result. Fu *et al*. (2013, p. 21) state “Taking the advantage of more WIM (Weigh-in-motion) data accumulated over the past decade, developing a new load requirement has become simpler and more quantitative.” WIM systems are typically deployed at long-span bridges, but appear appropriate for multi-span structures on high volume traffic corridors as well. Armored joints would lend themselves well to instrumenting for monitoring of the live load impact.

Recent heavy snowfalls in the New York City area, as well as over the entire East Coast have revealed the extreme vulnerability of armored expansion joints to plowing equipment, as illustrated in Fig. 3(b). A joint strong enough to sustain regular direct impact of a plow may not be cost-effective. Therefore a performance-based design should incorporate details avoiding such impact.

- Management

Joint failures are ultimately management failures. In a robust operation, no stage of the process or element of the product should inherit or transmit deficiencies to the others. The safe and cost-effective lifecycle of structures and networks requires redundancy, which in turn depends on continuity. In a typical bridge network, where 35% of the spans have expansion joints, the benefits of addressing these discontinuities before the need arises to address the entire structures are considerable.

Note: The material presented herein expresses the views of the author and not those of any organization.

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