QUANTITATIVE ASSESSMENT OF STRUCTURAL REDUNDANCY BY COMPUTER MODELING

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ABSTRACT

Structural Redundancy is a measure of viable alternate load paths available in case of failure of one critical member or component. It is an important design parameter for bridge structures as it affects the initial cost of construction, cost of maintenance over the life of the bridge, and in some cases, the perceived level of safety. A bridge that is deemed non-redundant is classified as fracture critical and is required to comply with special fabrication and testing requirements during construction and special inspection requirements during service. The latter is more significant in terms of lifecycle costs than the former. The increased maintenance requirements and the perception of increased risk to public have galvanized certain reluctance on the part of bridge owners to avoid superstructure types perceived to be fracture critical. Even though AASHTO (and other widely used design codes) provides a clear basis for the design of fracture critical bridges, the level of redundancy needed in a bridge superstructure to be classified "non-fracture critical" is not well defined or established. The classification criteria employed at the present time is mostly qualitative and is not based on a rational analytical determination. Further, geometric factors such as skew and curvature can interject considerable additional complexity and could make these qualitative classifications unreliable. The paper provides a general discussion of issues relating to redundancy, discusses a method for analytical determination of redundancy, and presents the results of a recent case study. The paper also discusses areas needing further refinements that would enable establishment of rational guidelines for ensuring uniform application of a rational redundancy criterion for highway bridges.

1. INTRODUCTION

Fracture critical structures are those where the failure of one critical tension element can lead to general failure of the structure. AASHTO Standard Specification (LFD) section 10.3.1 defines non-redundant load path members as those members where failure of a single element could cause collapse of the structure. Bridges consisting of one or more such non-redundant members are considered fracture critical. AASHTO LFD further indicates that flanges (tension implied) and web plates of one or two girder bridges as examples of non redundant load path members. Following this, one or two girder systems are generally considered as fracture critical regardless of a range of other factors that can have significant affect on structural redundancy, either positively or negatively, of a given bridge.

Specifically, the structural redundancy of a bridge is governed by the following two factors:

1. Residual capacity in the surviving longitudinal element(s) after suffering damage to one of the others[1], and

2. The capacity (strength and ductility) of transverse elements to shunt loads from the damaged longitudinal element to the surviving one(s).

[1] While progressive damage to other elements is relevant, simultaneous initial damage to multiple elements (as that can occur under accidental scenarios or intentional acts are typically not considered in classification of structural redundancy

The categorization based on the number of girders assumes that the only load paths available are the girder lines and no other viable alternate load path is available to prevent collapse of the bridge in the event of the failure of one girder. While this can generally be the case with simple span structures utilizing two I-girders, a multitude of other factors can beneficially or adversely affect the level of structural redundancy in a given bridge. These include:

- Girder cross section type and spacing
- Transverse elements (Cross frames & diaphragms)
- Presence of skew & curvature
- Span continuity
- Width to span ratio
- Ductility and ability to deform without collapse (Details, connections and material)

The term redundancy implies that there are alternate load paths available that are capable of preventing structural failure as a result of failure of a given critical tension element. The structures deemed deficient in such alternate load paths are termed 'fracture-critical'. As AASHTO and other widely accepted design codes provide a rational basis for the safe design of fracture critical bridges, this is not a significant design issue. In the US, fracture critical tension elements are required to meet certain special material and welding and testing requirements. The additional costs associated with this are typically less than 5% of the cost of fabrication[2], and is not a significant factor contributing to the overall cost of a bridge project. However, the fracture critical bridges are required to undergo frequent in-depth inspections and this can contribute to a significant difference in lifecycle costs of fracture critical bridges over those considered redundant. These factors and perhaps a general perception that fracture critical bridges represent some "increased risk" to the public has led to a general reluctance on the part of bridge owners to consider superstructure types perceived to be fracture critical.

Figure 1: The Storrow Drive Bridge (left) and South Station Ramps (right), Boston, MA

In many instances, the redundancy of a bridge superstructure is self evident, and is a matter of engineering judgment where no analysis is really necessary. However, in many cases, the traditional classification of redundancy based on the number of girders may classify some structures that possess sufficient redundancy as "fracture-critical". Under other circumstances, it may not necessarily be even conservative as many parameters (such as the global geometry

[2] In the case of the Storrow Drive Bridge (Figure 1), the added cost of fabrication for the tension elements classified as fracture critical was estimated to be around 5 c/lbs (based on a fabricator survey during design)

and deficiencies in capacity or detailing of transverse elements) could adversely affect the effectiveness of alternate load paths.

Thus in some situations, some quantitative determination of structural redundancy based on a rational analytical evaluation is warranted. However, before analytical methods can be applied to classification of redundancy, few issues need further resolution. Factors that are not clearly defined in the existing design guides include:

1. Performance Criterion: While the term “prevention of structural failure” easy to understand, is too broad of a performance criterion that lacks a precise definition. This can very well be project dependent and could be part of the design criteria.

2. Design Loading: Recognizing that the structure may be under some external loads at the time of damage or may be subjected to external loading in its damaged condition before discovery, it is logical to assume that the damaged structure should be able to carry some live loads over and above its self weight before reaching “structural failure” defined in item 1 above.

3. Damage Condition: The design damage condition that the structure must be able to sustain before reaching the failure defined in item 1 under loading in item 2. Recognizing the probabilistic basis inherent in typical design requirements, the selection of this damage condition must reflect reality in terms of improvements in materials, fabrication procedures and the likely hood of discovery before reaching the design damaged condition.

In short, recognizing redundancy as an optional design parameter to be met, it should be possible to define how much redundancy is needed in a structure to be considered non-fracture critical. This would enable the design process to include sufficient redundancy enhancing measures that would bring the resulting structure up to the desired minimum level of performance expected under a damaged scenario. The redundancy enhancing measures could include providing excess longitudinal carrying capacity (than needed by standard design), providing additional transverse elements (often a key element in alternate load paths), limiting girder spacing, and in extreme cases, providing split tension elements. As redundancy is really a life cycle and acceptability issue, the formation of rational guidelines relating to redundancy would enable rational decision making on a project by project basis.

2. QUANTIFICATION OF STRUCTURAL REDUNDANCY

As stated previously the first difficulty faced in assessing the structural redundancy is the lack of consensus and clear definitions on several key issues. On the other hand, there is consensus on what structures are routinely considered as redundant (or having sufficient redundancy to qualify as non-fracture critical). The study described in NCHRP Report 406 *Redundancy of Highway Bridge Superstructures* uses this concept and statistical analysis to define minimum criteria met by bridges that we routinely consider redundant, and has proposed the same as the measuring criteria for meeting sufficient redundancy. The method is well suited for analytical application and can be considered as the first comprehensive attempt to propose universal criteria quantifying the redundancy of bridge structures using a limit state approach. In formulating the redundancy criteria, it first defines the following limit states:

- Member Failure limit state (LF_1) – when the member reaches its design capacity in elastic method of analysis (without damage)

- Ultimate limit state (LF_u) – when the ultimate capacity is reached by developing a failure mechanism (without damage)

- Functionality limit state (LF_f) – when the live load deflections reach 1/100 of span length (without damage)

- Damaged condition limit state (LF_d) – when the ultimate capacity of the system is reached in the damaged condition

Recognizing that the reserve structural capacity available to carry live loads prior to reaching these limit states can be quantified by the difference (R-D), where R is the structural resistance and D is the dead load effect, the limit state factors LF_1, LF_u, LF_f and LF_d are defined as the scale multiplier that must be applied to two side by side HS20-44 trucks in order to reach the above limit states. These can be found by incremental analysis solution that considers material and geometric non-linearities in the system. Using the above limit state factors, the following system reserve capacities are defined:

- System reserve ratio for ultimate capacity, $R_u = LF_u / LF_1$

- System reserve ratio for functionality limit, $R_f = LF_f / LF_1$

- System reserve ratio for damaged condition, $R_d = LF_d / LF_1$

Through probabilistic analysis using a large number of girder bridges that are clearly considered to provide sufficient system redundancy (such as four girder bridges), minimum acceptable values of the system reserve ratios are determined to be $R_u > 1.30$, $R_f > 1.10$, and $R_d > 0.50$. This provides a quantifiable basis for evaluating redundancy of a given structural system.

However, a review of system reserve ratios for the ultimate capacity and functionality limit would show that they are not directly related to redundancy, and can be expected to be automatically satisfied if the existing code provisions are correctly followed for the strength design and serviceability checks. Thus, the only relevant parameter for addressing the structural redundancy aspect (in author’s opinion) is the system reserve ratio for the damaged condition. In the study described in the following section, a 3-dimensional finite element model was used to obtain the member failure limit states (LF_1) and the damaged condition limit states (LF_d) for the different bridge layouts examined so the system reserve ratios (R_d) could be established.

3. CASE STUDY – TWIN BOX GIRDER SUPERSTRUCTURES

Steel box girder superstructures are used effectively in most roadway geometric configurations and span layouts. The number of boxes used in cross section varies from single, twin and multi-box arrangements. In comparison to I-girders, box girders possess considerable capacity for load re-distribution due to the inherent torsional stiffness. While many consider single box configurations as fracture critical and three box configurations as redundant, there is considerable lack of consensus with respect to the twin box cross sections. Thus in addressing structural redundancy of box girder bridges, the twin box section is a logical baseline.

In some instances the twin-box systems are classified as fracture critical (when the only parameter considered is limited to the number of girders), ignoring their ability for load re-distribution under most realistic damage scenarios. This has resulted in considerable variability in design treatment of twin steel box girder bridges. Their relatively wide-spread use is another factor for selecting the twin box cross sections for the present case-study demonstrating the analytical procedure as well as the role of other factors affecting structural redundancy discussed before. A simple span twin box configuration was selected as this could be defined as the baseline case in a wider treatment of box girder bridges of all types. The case study also looked into the effect of horizontal curvature, as this is a key factor that could negatively affecting structural redundancy.

The cross section amused in the present study is shown in Figure 2. The 34-ft wide roadway (31-ft curb to curb) is supported by two 9-ft deep steel box girders (Girders A and B). A 225-ft long simple span arrangement was selected as simple spans are more critical and basic. The simple spans studied presently also approximate the redundancy of end spans of continuous bridges. As continuity provides additional redundancy, any requirements established can be relaxed for interior spans as appropriate.

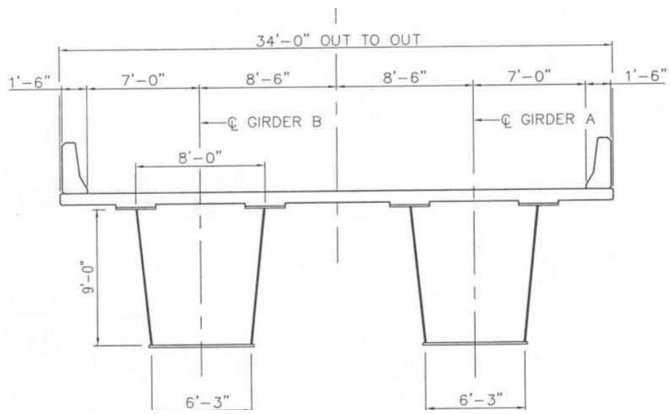


Figure 2: Typical Cross Section

As previously discussed, horizontal curvature is a key parameter that affects redundancy of a two box system. With curved alignments, the inner box (Girder A) sheds loads to the outer box (Girder B) requiring significantly more capacity in Girder B with respect to Girder A. As a result, failure of Girder B is more critical to the overall failure of the bridge, as the proportion of the resulting lost capacity is more significant than in a tangent alignment where both boxes are equal.

The baseline structure was selected with a tangent alignment (no horizontal curvature). The structural framing included internal cross frames and top lateral bracing. Four equally spaced full depth intermediate diaphragms were provided between the two boxes in addition to the end diaphragms at the bridge abutments. To examine the effect of curvature, the alignment was modified to reflect three separate levels of curvature keeping the simple span arrangement (Figure 3).The curvatures examined included R = 1150-ft, 750-ft and 400-ft. First the steel boxes were designed for dead and live load effects as described in the following.

Dead Loads: Dead load analysis simulated the bare steel erected with both steel weight and concrete slab dead load applied as a non-composite load to the steel framing. Then the slab was assigned the proper stiffness properties to simulate the composite steel-concrete slab interaction. The superimposed dead loads such as the wearing surface and barriers were applied to the composite bridge. This defines the Dead Load Model (DLM).

Live Loads: Two HS-25 lanes of live load were positioned to create maximum bending / shear / torsion in the steel boxes.

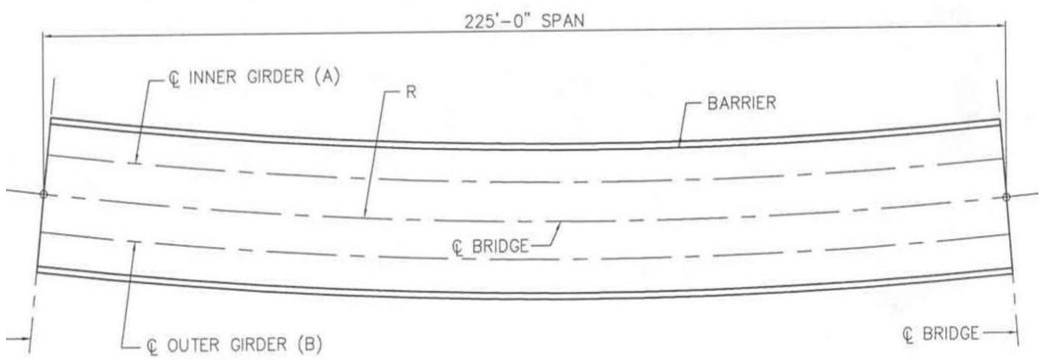


Figure 3: Plan Layout

Box Design: The steel plate sizes of the two boxes were determined by following AASHTO LFD and Curved Girder Specifications as appropriate considering factored loads and stresses. The resulting plate sizes and framing plan are shown in Appendix 1. The design of curved alignments required iterative analysis due to shedding of loads from interior to the exterior box. This is illustrated in Table 1 for the bottom flange of the box girders.

Table 1: Effect of Curvature on Load Sharing Between Girders A and B

Alignment and/or Curvature		Girder A		Girder B	
		t_{bf} (in)	σ_{bf} (ksi)	t_{bf} (in)	σ_{bf} (ksi)
Tangent		1 1/8	50.0	1 1/8	50.0
R = 1150	Trial 1	1 1/8	46.1	1 1/8	54.1
	2	1	44.3	1 3/8	51.0
	Final	7/8	44.1	1 1/2	49.0
R = 750		7/8	42.9	1 1/2	49.8
R = 400		7/8	37.9	1 5/8	49.0

Where
t_{bf} = Bottom flange plate thickness (inches)
σ_{bf}= Bottom flange factored stress (ksi)

As indicated in Table 1, the girders of the bridge with the tangent alignment are the same. The box sizes for the tangent alignment were used as the starting point for the curved alignment with 1150-ft radius (Trial 1). The stresses obtained with Trial 1 show a reduction in the stress level in Girder A and an increase in the stress level in Girder B relative to the tangent alignment. The plate sizes were refined (Trial 2) and the bridge was re-analyzed to obtain the new plate stresses. This process was repeated until an optimal solution was obtained. The same design procedure was used for the other two radii as well. The 7/8" bottom flange for Girder A is governed by the minimum thickness considerations and not the level of stress.

The Finite Element Software package ADINA was used for the modeling. The model used in this preliminary study can be considered coarse in some aspects, and the slab was modeled as a linear elastic material (cracking of concrete slab was not modeled).

Fracture Analysis: The damage scenario considered in the study consists of complete sudden fracture of the bottom flange and the webs of the outer box at the mid-span location (most critical). The key aspects of behavior that needs to be captured for realistic simulation of bridge response to this damage scenario includes:

- Pre-existing stress condition followed by instantaneous release of steel stresses at the fracture location
- Material response (Yielding and stress hardening of steel, cracking of concrete)
- Dynamic response
- Effect of “non structural” elements such as barriers
- Stresses due to live loads on the damaged structure

Pre-existing stress condition & Fracture - Starting with the dead load model (DLM), the bottom flange of the mid-span section of Girder B was fractured. With linear elastic analysis, under DL alone, the failure resulted in stress conditions exceeding the yield stress of 50-ksi in the surviving girder. This indicated that the considered failure (of one girder) would result in yielding of the structure at other critical locations. This yielding and permanent deformation is an integral part of the load-redistribution following the simulated fracture. To capture this behavior, the DLM model was revised to include non-linear elastic-plastic material behavior for all steel elements.

Dynamic Response (Impact) – Bridge superstructure response to sudden failure of one girder would induce an initial transient response before reaching the new static equilibrium condition. This dynamic response results in time variant deflections and stresses in excess of the final static values. Typically, transient dynamic effect is not directly included in analysis, but its effects are represented by a suitable “impact factor” applied to the static results. The magnitude of the transient dynamic response as a fraction of the static (and hence the “impact factor”) for a linear elastic system to an instantaneous loading (assuming no system damping) can be shown to be 100%. However, the impact factor for the present application would be considerably smaller due to the following factors:

1. Structural damping and additional energy dissipation due to yielding of steel and cracking of concrete
2. Finite duration event vs. “Instantaneous”

While the reduction of impact factor due to typical levels of structural damping is relatively minor, the additional energy dissipation due to yielding of steel and cracking of concrete is expected to have a very substantial effect on the impact factor. In addition, the progressive yielding and other damage associated with load re-distribution following the initial fracture makes the event “non-instantaneous”, or of finite time duration. The other key issue with respect to the application of impact factor is the bridge loading assumed immediately prior to the fracture. Specifically, it could be debated if the impact factor should only be applied to the DL stresses only, or to the total loading including the LL corresponding to the damaged condition limit state (LF_d). As the live loading on the structure at the time of failure is likely to be considerably less than that corresponding to LF_d, and it is likely that the dynamic stress would have been dissipated before the structure is subjected to increased LL, application of the impact factor to the total loading appear overly conservative. This issue requires further study before a reasonable approach can be formulated. For the purpose of the case study, the impact factor was ignored. To compensate for this effect, the positive contribution from non-structural elements (barriers) was ignored. In a typical bridge, barriers would provide considerable additional carrying capacity, global stiffness and thus reduce the level of dynamic response.

Material Response - The stress distribution following fracture results in widespread yielding of the surviving girder and local sporadic yielding of transverse elements. The deck stresses indicate that there will be cracking of the deck slab. The yielding and stress hardening of steel was modeled using the stress-stain curve shown. Complete failure was assumed when steel strain reached 16%. The deck response was approximated by a linear elastic material in this preliminary study.

None of the models resulted in overall failure under self weight. However, large scale yielding and significant displacement occurred following the simulated fracture. Figure 5 show the service level DL stress pattern before and after failure for 1150-ft radius indicating the extent of yielding after fracture.   The deflections at the critical section resulting from the fracture are given in Table 2.

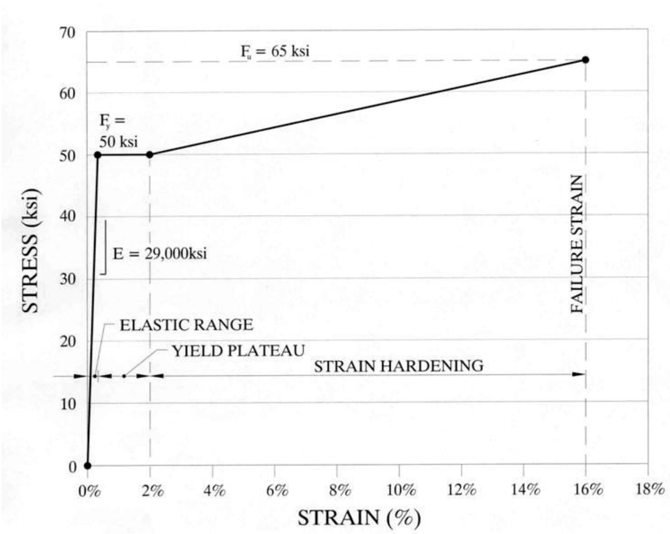
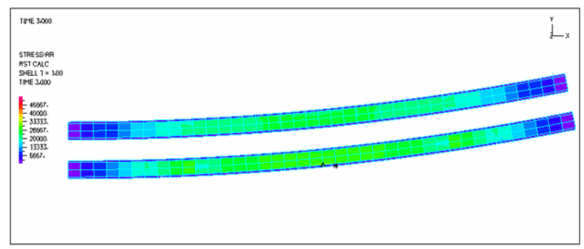


Figure 4: Assumed stress-strain model for steel

(a) Before



(b) After

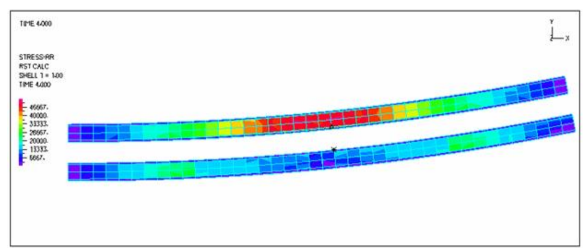


Figure 5: Bottom flange DL stress distributions before and after fracture

Table 2: Deflections under Self Weight

Bridge Layout	Deflections Under Dead Loads (in)			
	Before Fracture (Camber)		After Fracture	
	Non-Comp.	Total DL	Total	Additi-onal
Tangent	14.4	16.5	22.6	6.1
1150' R	14.7	16.9	25.6	8.7
750' R	16.1	18.7	27.6	8.9
400' R	20.6	24.0	34.0	10.0

System Reserve Ratios – Only the System Reserve Ratio for the Damaged Condition (R_d) described in NCHRP Report 406 was obtained (discussed). This is the only system reserve ratio applicable to ensuring adequate reserve capacity in the damaged condition. As noted previously, it was assumed that the current standard design requirements for strength and serviceability would ensure satisfactory System Reserve Ratios for Ultimate Capacity (R_u) and for Functionality Limit (R_f) as these are defined for the undamaged condition of the bridge.

As outlined in NCHRP Report 406, two HS-20-44 trucks were placed side by side at the critical section of the undamaged bridge, and the resulting live load stresses were obtained. The Member Failure limit state (LF_1) was computed as the multiplication factor that must be used on the prescribed live loading to raise the service level steel stress under combined Dead and Live Loads to 50-ksi (when the member reaches its design capacity) in the elastic method of analysis (without damage).

The damaged condition limit state (LF_d) was obtained by incremental live load analysis of the fractured bridge models accounting for the nonlinearities of behavior. The HS-20 truck live loading on the models were increased until the models could not support any additional load increments or the 16% max steel strain was reached. The assumed stress-strain relationship for the nonlinear material model for steel is shown in Figure 4. As seen from the figure, the material model is multi-linear and simulates both yielding as well as strain-hardening behavior. The incremental live load vs. bridge deflection plots are shown in Figure 6.

It was generally observed that the bridges could sustain rather large deflections before the failure conditions were reached. However, the rate of increase in bridge deflection with increased live load was observed to increase beyond a certain level of loading, indicating that the instantaneous system stiffness is reduced with increased loading. Considering the above two factors in combination (large deflections and the softening behavior), it was decided to examine the evolution of this instantaneous stiffness and use a conservatively selected useful LL capacity limit that is lower than the collapse load.

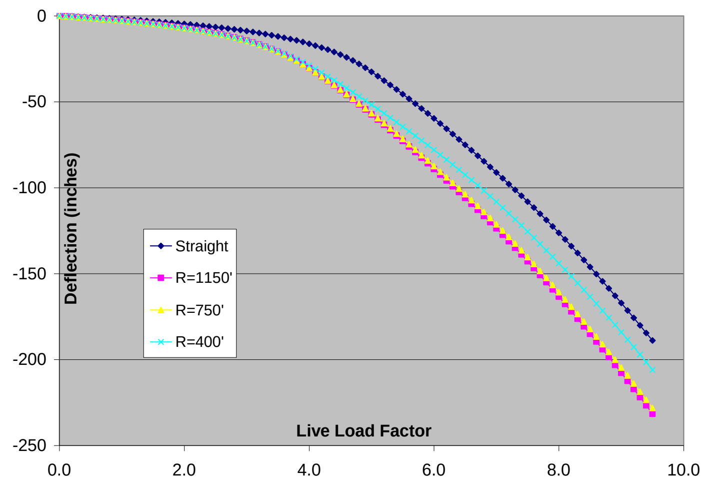


Figure 6: Incremental LL vs. Bridge Deflection

Figures 7 show the evolution of bridge flexibility and stiffness with increasing load. The general yield plateau and the strain hardening regions are clearly visible in these plots. With increasing live load, the bridge stiffness gradually decreases up to a certain point. Then as general widespread yielding begins to occur, the stiffness decreases until it reaches about 70 kip/in (=0.015 in/kip flexibility). Beyond this point, the rate of change in stiffness increases indicating that strain hardening has begun to occur. For the purpose of this study, the maximum LL capacity was taken conservatively as that corresponding to 70 kip/in stiffness line, even though the bridge model continues to support considerably more load until the critical section of the surviving box reaches the failure strain of 16%.

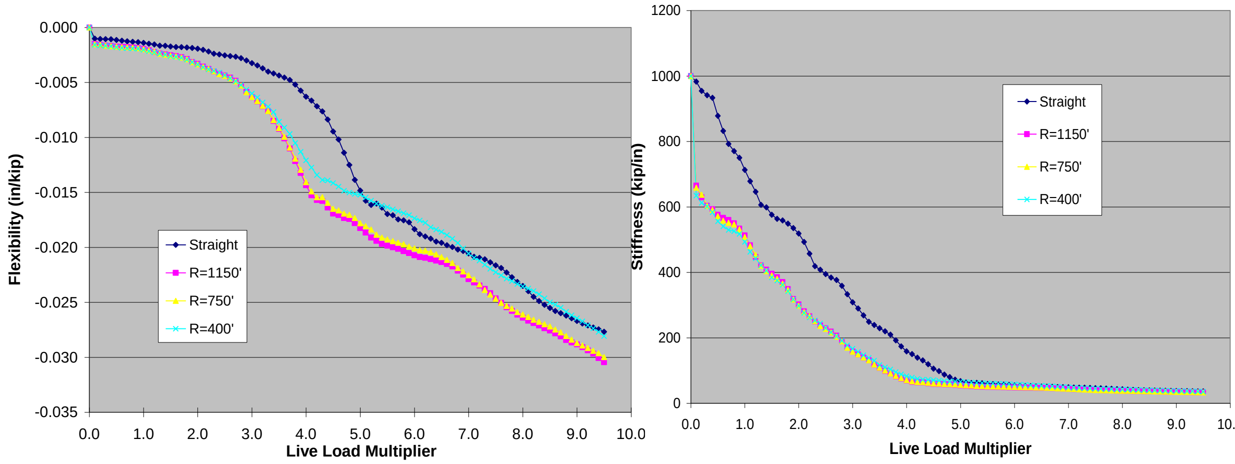


Figure 7: Instantaneous Flexibility and Stiffness vs. Total LL on Bridge

Using the criteria proposed, the redundancy factors for the four bridges were determined as shown in Table 3. These factors indicate that significant reserve capacity exists in these bridges, enough so that they would be considered redundant as per the guidelines of NCHRP Report 406.

Table 3: System Reserve Ratio for Damaged Condition

Bridge Layout	Redundancy Analysis		
	Limit States		
	LF_1	LF_d	R_d
Tangent	5.6	5.0	0.89
1150' R	6.6	4.1	0.62
750' R	6.4	4.1	0.64
400' R	6.8	4.7	0.69

It must be noted that while the Limit State factor LF_d reported above is taken somewhat subjectively, it is still a conservative value as the damaged bridges can support load increments well beyond the limits used. It is noted that all the bridges meet the minimum System Reserve Ratio for Damaged Condition (R_d) of at least 0.5 per the NCHRP Report 406 for meeting the requirement for non-fracture critical and redundant structures.

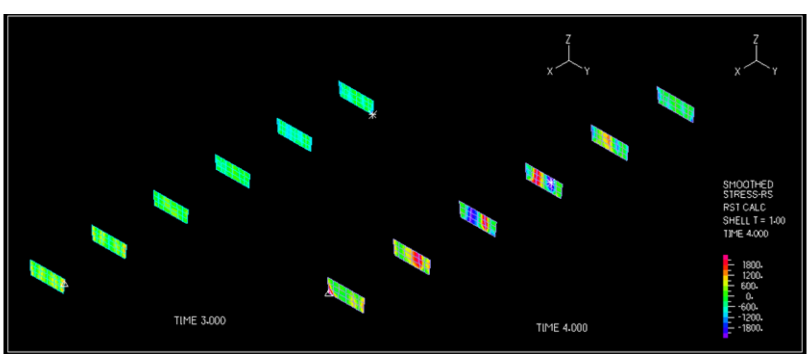


Figure 8 shows the shear forces in the cross diaphragm between the boxes before and after fracture. It indicates that these diaphragms could be the key to the redundancy of the overall structural system.

Figure 8: Shear in diaphragms before and after fracture of bottom flange of one girder

4. AREAS NEEDING FURTHER STUDY & DEVELOPMENT

Following are some areas that could substantially benefit from further examination and investigation with respect to bridge redundancy criteria and analysis.

1. Minimum reserve capacity requirement for redundancy
2. Application of the standard live loading
3. Dynamic effects – application of impact factor
4. Deflection limits in the damaged condition
5. Design damaged condition

Minimum Capacity Requirements for Redundancy: The NCHRP 406 minimum requirement of $R_d = LF_d / LF_1 > 0.5$ for sufficient redundancy was arrived at by statistical analysis of existing bridges clearly accepted within the industry as redundant. A review of the limit states LF_1 and LF_d indicate that this requirement, in a border sense, imply that the damaged bridge superstructure should at least have 50% of the original reserve carrying capacity for live loads (beyond the capacity to carry its self weight).

Taking the AASHTO LFD minimum design strength required in the pre-damaged condition as:

$$1.3\ (DL) + 2.17\ (LL+I)$$

The minimum capacity requirement for the damaged bridge can be approximately stated as:

$$1.0\ (DL) + 0.5*[1.3\ (DL) + 2.17\ (LL+I) - 1.0\ (DL)] = 1.15\ (DL) + 1.08\ (LL+I)$$

Thus, the NCHRP 406 requirements amounts to requiring the damaged bridge to be able to carry its self weight and the full design live load with some reduced factors of safety. While this is certainly fulfils requirements for a redundant structure, there is some potential for it to be overly conservative due to the following reasons:

1. The full design LL rarely occur in reality
2. The DL deflections associated with a full fracture of a longitudinal carrying element would make it readily detectable and some interim measures to limit live loading would be taken before a full repair is effected

Further study in this area could lead to some relaxation of the minimum redundancy requirements

Application of the Standard Live Loading: While the application of the standard live loading per NCHRP 406 to determine the different limit states works well for the global longitudinal girders of the superstructure, the concentrated load effects due to the application of scale multipliers of the truck loading could produce unrealistic demand conditions for the local transverse elements. As these local transverse elements are key to providing alternate load paths, the artificially high local load conditions could cause pre-mature failure of the transverse elements and thus provide an artificially low overall redundancy rating. This issue warrants some further study.

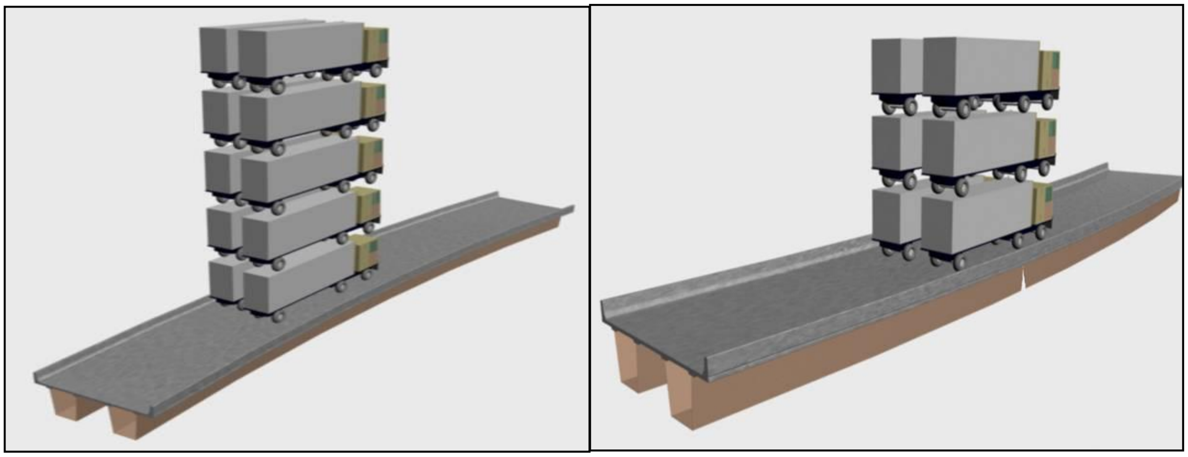


Figure 9: Loading for member failure limit state, LF_1 (left) and damaged condition limit state, LF_d (right).

Max. Deflection Limits for Damaged Condition: The system reserve ratio for the damaged condition is basically a residual capacity requirement, and the case study results show that when the damaged bridge accumulates large structural deformations (reaching approx. 10% of span length) under the incremental live loading process. As this level of deflections would render the bridge unsuitable for use, a meaningful deflection limit could be combined with more realistic minimum reserve capacity requirements (see item 1 above) to provide a more meaningful requirements for structural redundancy.

Dynamic Effects: A discussion on issues relating to the dynamic response under sudden failure was provided previously in section 3. The key issues needing further study using a sophisticated transient dynamic model are:

- Recommended effective impact factor (function of)
    - Effective damping considering the energy dissipation due to yielding and other permanent damage
    - Duration of the damage event

- Recommended application assumptions. That is the assumed loading condition immediately before the initial damage and what impact factor (if any) to be applied for subsequent load increments

Design Damaged Condition: One entire box section out of the twin boxes was assumed damaged in the case study. However, considering the advances in material and welding technology, this damage scenario could be overly conservative. Development of some guidelines for more realistic damage conditions that considers the specific conditions and the properties of the superstructure would be highly beneficial.

5. CONCLUSIONS

Following are the major conclusions that can be drawn from this study:

1. The redundancy classification based on the number of girders is not reliable as many other factors affect the capacity of alternate load paths and the spanning capacity of the damaged bridge.

2. For redundancy, the capacities of the transverse elements are as important as the remaining longitudinal spanning capacity of the damaged superstructure. Other key factors include the global geometric properties such as skew and curvature.

3. As well accepted design codes provide for safe design of fracture critical bridges, it is possible to treat redundancy as a life-cycle cost issue

4. NCHRP 406 provides a basis for analytical determination of structural redundancy of a given bridge through computer modeling. This procedure was applied to a 225-ft simple span twin box-girder superstructure. The analysis showed that the bridge layout studied met the redundancy requirements outlined in NCHRP 406.

5. Following are beneficial design / detailing factors that would enhance redundancy of steel girder bridges:
    - Continuity
    - Number of girders and girder spacing
    - Diaphragm and bracing between girders and their detailing
    - Minimization of the difference in strength and stiffness of the girders in horizontally curved alignments
    - Split tension elements (in extreme cases)

6. There are several areas that could benefit from further study. These include:
    - Redundancy criteria
    - Application of the standard LL
    - Deflection limits in the damaged condition
    - Quantification of dynamic effects
    - Design damaged condition to be considered

7. It is felt that a more comprehensive treatment of redundancy of box girders in general and the development of design guidelines are possible. One possible approach may be to define a baseline system with features that meets redundancy requiems and provide guidance on how to tighten or relax the baseline requirements depending on the project specific features such as the number of girders, continuity, curvature & skew, etc.

6. REFERENCES

[1] AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, 2003. American Association of State Highway and Transportation Officials, Washington, D.C.

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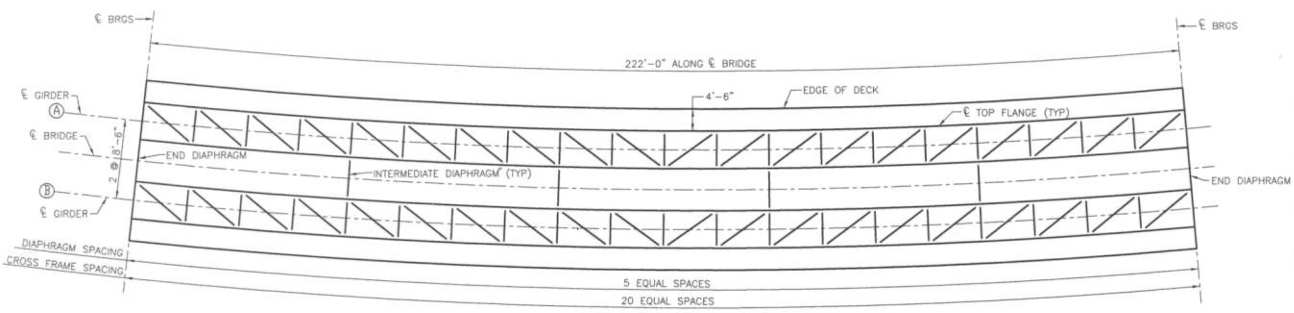
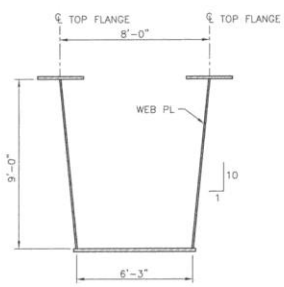
APPENDIX 1



FRAMING PLAN



TYPICAL GIRDER SECTION

PLATE SIZES – MIDSPAN SECTION				
		TOP FLANGE	BOTTOM FLANGE	WEB
GIRDER A	STRAIGHT	20 x 1 1/4	75 x 1 1/8	108 x 3/4
	R=1150'	20 x 7/8	75 x 7/8	108 x 3/4
	R=750'	20 x 7/8	75 x 7/8	108 x 3/4
	R=400'	20 x 7/8	75 x 7/8	108 x 3/4
GIRDER B	STRAIGHT	20 x 1 1/4	75 x 1 1/4	108 x 3/4
	R=1150'	24 x 1 3/4	75 x 1 1/2	108 x 7/8
	R=750'	24 x 1 3/4	75 x 1 1/2	108 x 7/8
	R=400'	24 x 1 7/8	75 x 1 5/8	108 x 1

PLATE SIZES – END SECTION				
		TOP FLANGE	BOTTOM FLANGE	WEB
GIRDER A	STRAIGHT	20 x 7/8	75 x 7/8	108 x 3/4
	R=1150'	20 x 7/8	75 x 7/8	108 x 3/4
	R=750'	20 x 7/8	75 x 7/8	108 x 3/4
	R=400'	20 x 7/8	75 x 7/8	108 x 3/4
GIRDER B	STRAIGHT	20 x 7/8	75 x 7/8	108 x 3/4
	R=1150'	24 x 1 3/8	75 x 7/8	108 x 7/8
	R=750'	24 x 1 3/8	75 x 7/8	108 x 7/8
	R=400'	24 x 1 1/2	75 x 1	108 x 1