

EVALUATION OF AGING REINFORCED CONCRETE (R/C) BRIDGE GIRDERS

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ABSTRACT: *Among majority of pervasive mechanisms, visual condition monitoring is predominantly used for structural condition assessment. Outcomes of this mechanism are used for future forecasting, but not considered fully reliable. This paper provides a methodology to facilitate asset owners in informed decision making, by supplanting the visual condition data. Methodology looks at the time series behaviour of reinforced concrete girder, due to load induced flexural stresses, shrinkage and creep. Proposed time dependent procedure will provide sufficient lead time to asset owners for proactive measures.*

KEYWORDS: Aging, R/C Bridge Girders, Time Series Behaviour, Time of Intervention, Asset Owners

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1 INTRODUCTION

Bridges contribute a lot in economic and social growth of any nation; hence, it becomes a great responsibility to capitalise on bridge's long term durability, during its in-service life. At present, average age of an in-service bridge in USA is 43 years [1], where, as per National Bridge Inventory 12.36% of bridges are rated structurally deficient and functionally obsolete [2]. Around 60% of bridges in Australian are estimated to be approximately 50 years old as well [3]. Likewise, 40% of Canadian bridges are also older than 50 years [4]. In New Zealand, average life of bridge is considered to be around 40 years old too [5]. Therefore, it is safe to assume that the majority of developed countries bridge infrastructure is around 40 years old and has reached 53.33% of designed life, if designed for maximum 75 years.

Above mentioned are derived from the dataset of bridges, maintained after conducting regular bridge inspections. Regular bridge inspections are globally known as "visual inspections", take place annually, biannually and once in two years, depending on the road authority and organisations [6-7]. Visual inspection data is either recorded in words, such as, good, fair and bad, or in numbers, ranging from 0 to 9, where 0 stands for poor and 9 stands for good. Such qualitative dataset leads towards "subjective" outcomes. According to Catbas et al. & Soliman et al. [8-9], beyond 50% of visually inspected bridges can be considered misclassified. Failure of I-35 W Mississippi River Bridge is a classic example, which took place due to delay in intervention and as a consequence causing an unfortunate catastrophic failure.

Majority of in-service aging bridges are reinforced concrete (R/C) bridges. The number of such assets is increasing at an alarming rate in local and regional councils and shires. These bridges were constructed during early 50's till late 60's. Visual inspection data of these bridges is very scarce too. Hence, they have started becoming a concern for the respective asset owners.

In order to prevent any catastrophic failure or premature accidents, the authors' aim is to provide a time dependent methodology, which evaluates an aging bridge girder. It identifies the right time of intervention, so that, preventive measures can be taken to preserve bridge girder for longer time periods, supplant informed decision making and enhance forecasting, such as capital works program and maintenance program. This paper limits its discussion to R/C girders. Time dependent evaluation deterministic analysis is taken into account, considering increasing traffic live load along with shrinkage and creep. Output of the analysis is presented as crack widths resulting due to increase in flexural stresses. To demonstrate the analysis, authors have considered "U" slabs, which

act as bridge girders designed in early 1960s. These slabs were designed to cater H20 – S16 live loading but now cater S1600 and M1600 traffic live load. Impact of this increment is demonstrated in this paper.

2 ASSET OWNERS AND RELATED CHALLENGES

Managing bridges is a very challenging task and comes with wealth of responsibilities. Asset owners are the group of people who take up this role and try to execute with full precision and accuracy, so that they can circumvent any unexpected mishap. Asset owners are classified as per the region, such as, state road authorities, local and regional councils and shires. VicRoads, a state road authority of Victoria manages approximately 6,000 bridges and culverts [10]. In addition, Victoria's 79 municipal councils also maintain a similar number of assets.

It is not considered challenging for state asset owners to maintain a credible inventory of 6,000 bridges and culverts, inclusive of, visual inspection condition reporting and maintenance history, because of the availability of all required resources. On the other hand, rural councils encounter some limitations in the development of such credible inventories, for example,

- Short supply of competent and experience visual inspectors.
- Lack of credible information, such as, as-built information.
- Poor record keeping of maintenance history.
- Quality of available technical information.
- Maintaining the changing regime of data collection.
- Increase in traffic loads.
- Budgetary constraints.

An accurate and reliable inventory is enormously valuable for an asset owner, since, it is effectively used to evaluate the present condition of bridges, forecasting the right of intervention and developing maintenance and capital works program [11].

On the other hand, it is challenging for the visual inspectors to record the time dependent effects of changing traffic conditions, material properties and surrounding environment, because the consequences of these varying parameters are hard to conceive through a naked eye. Hence, this influences the reliability of recorded data and affects the decision making. Such phenomenon, where time dependent effects remain unnoticed is also seen in the inspection reports made available to research team by a regional council.

2.1 TRAFFIC LOADING IN AUSTRALIA SINCE 1960

Traffic load is comprised of cars, buses and trucks, but authors limit the discussion to trucks alone. Truck sizes have increased significantly in Australia, since 1960's, due to the substantial growth in demand of consumer and industry. Henceforth, legal load limit allowances on bridges have taken a dramatic shift; such an illustration is provided in Figure 1. Australia has gone through three main regimes of loading criteria. At present, Australian Standards [12] recommend to use either S1600 or M1600 truck loading for new bridge design. Equivalent loading is considered to evaluate the present condition of aging bridges; designed prior to the introduction of Australian Standards, 2004. Regular evolution in legal load limit is expected to be one major factor which affects the performance, durability and enhances deterioration of bridges [13, 14]. Figure 1 illustrates such a growth in traffic loading, since 1960 till 2010, in Victoria, a state of Australia. Primary axis illustrates the load imposed by the group of axles, whereas, secondary axis defines the uniformly distributed load used in conjunction with the point load.

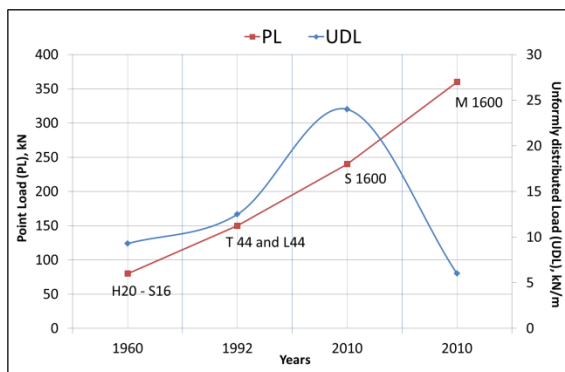


Figure 1: Truck loading since 1960 till 2010, as per NAASRA, AustRoads and Australian Standards, used in Australia

3 TIME DEPENDENT ANALYSIS THEORY

Time dependent quantitative assessment of an aging bridge girder is a complex phenomenon, as compared to the qualitative assessment. In qualitative assessment, subjective recordings are utilized effectively; using markov models and reliability methods, to perform time dependent analysis. However, for quantitative assessment, it is essential to have the inventory consisting change in imposed live loads, sustained dead loads, shrinkage and creep with time. But, it is challenging to record all these parameters on a regular basis, through visual inspections; therefore, such data set is barely maintained and available, especially at regional

level. Therefore, authors propose a time dependent methodology to do such analysis.

3.1 METHODOLOGY

A bridge girder is always designed for higher live load than required. It is achieved, either using working stress method or limit state criteria. Nevertheless, as a girder ages its structural capacity reduces with time, due to degradation in material properties, excessive usage of the asset or corrosion setting at tensile reinforcement level. However, it is hard to determine a single predominating cause which accounts for degradation because each bridge girder has its own variability. Such as, location of the bridge, type of traffic its growth with time, how aggressive the surrounding environment is and degradation rate of material properties. Of them all, increase in traffic can be regarded as one common dominant parameter which affects the bridge girder's durability, because legal load limits have increased significantly over the past few years, as shown in Figure 1. Such trend accounts for slow but constant degradation of the girder, if not maintained or intervened at appropriate time. Consequences of such growth can be noticed in the form of flexural cracks, which appear at soffit and reinforcement level of the girder.

Cracks in R/C girders can develop from an early age, due to shrinkage, creep and handling during construction, which are hardly visible to the human eye. But, over time as concrete matures and the load imposed on the girder increases, crack widths increase and become visible. Later such cracks set corrosion in motion, further degrading the strength of girder and triggering a vicious cycle of stiffness. Several researchers and standards recommend a critical crack width range between 0.3mm to 0.41mm, to keep as a benchmark. But, these cracks never get documented in regular inspection reports, as they are too fine and hard to conceive from a distance. In addition, such cracks have the tendency of opening and closing, due to variability in loading and plasticity of concrete. Therefore, it's difficult to document them until the crack width reaches a higher value.

Therefore, to eliminate such inconsistency, authors propose a theoretical model which calculates crack width with time, resulting from induced flexural stresses due to increase in live load. It empowers the owners to predict the crack width at appropriate time, without completely relying on the visual condition inspectors to detect them. Asset owners can decide the time of intervention based on the outcomes of the analysis and hierarchy of their bridge assets. Authors define right of intervention as, Theoretical Age of Intervention (TAoI), which predicts cracks within a girder reaching a structurally critical stage, which may not be necessarily visible to human eye. Here structurally

critical stage doesn't relate to critical crack width stage. A structural critical stage can be decided by an asset owner or a community of experts within that region. To identify TAOI, the model requires built details of the girder, including, structural dimensions, number and type of reinforcement used, concrete strength, location of bridge and loading used to design the girder. With these inputs, each girder can be assessed for the increase in traffic loading since its design and construction time, for example, a bridge constructed in 1960's can be analysed as per Figure 1. Another main aspect of the model is positioning of the axle loads on the girder to generate maximum bending moment, under serviceability conditions, without using limit state functions.

3.2 CRACK WIDTH ASSESSMENT AS PER RESEARCHERS AND CODES

In this study, several crack width formulas, proposed by researchers and suggested by codes for flexural members, are used to compute the analysis. Authors are providing a brief description of these formulas in succeeding sections. For in-detail explanation of any formula, cite to the reference listed in section. Also, majority of the formulas are empirically driven and sensitive to unit conversion, therefore authors use SI units throughout the discussion.

3.2.1 R. I. Gilbert and G. Ranzi

Gilbert and Ranzi [15] proposed crack width prediction formula based on Tension Chord Model and performed laboratory experiments for 400 days, for a total of 12 simply-supported beams and one-way slabs.

$$(w^*)_{soffit} = \frac{k_{cover} s^*}{E_s} \left[\frac{T}{A_{st}} - \frac{\tau_b s^*}{d_b} (1 + n_e * \rho_{tc}) - (\epsilon_{sh} * E_s) \right] \quad (1)$$

$(w^*)_{soffit}$ = final maximum crack width at the member soffit, k_{cover} = term to account for the dependence of crack width on the clear concrete cover, c . E_c = elastic modulus of concrete and $\Phi(t, \tau)$ = creep coefficient. Both parameters are time dependent and can be calculated using clause 3.1.2 and 3.1.8 of [16], respectively. ϵ_{sh} = shrinkage strain which may be computed using clause 3.1.7 of [16] and a time dependent value. $n_e = E_s/E_e$ effective modular ratio. d_b = nominal diameter of a bar. T = resultant tensile force estimated using A_{st} = area of steel in tensile zone and M_s = in service bending moment. τ_b = bond shear stress between the steel and the surrounding tensile concrete.

3.2.2 S. H. Chowdhury and Y. C. Loo

Chowdhury and Loo [17] presented an elementary and precise formula to calculate crack widths, after conducting a sequence of tests on full-size reinforced and partially prestressed concrete beams.

$$w_{cr} = \left(\frac{f_s}{E_s} \right) * \left[0.6 * (c - s) + 0.1 * \left(\frac{\Phi}{\rho} \right) \right] \quad (2)$$

w_{cr} is the average crack width, f_s = stress in reinforcement at service load, E_s = elastic modulus of steel, c is the concrete cover, s accounts for average spacing between reinforcing bars, Φ is the average bar diameter and ρ is the reinforcement ratio.

To obtain maximum crack width, w_{max} , Chowdhury and Loo suggest to multiply the w_{cr} , average crack width with 1.5, i.e. $w_{max} = 1.5 w_{cr}$.

3.2.3 Gergely and Lutz

Gergely and Lutz [cited in 18] suggested crack width formula based on the statistical analysis of experimental test outcomes, at the tension reinforcement surface of the flexural member. Most of the experimental test results were taken from other researchers, such as, Hognestad, Clark, and Kaar and Mattock [cited in 18].

Proposed equation is as follows:

$$w_s = 0.011 \sqrt[3]{c * A_o} (f_s - 34.45) * 10^{-3} \quad (3)$$

w_s is the most probable maximum crack width at level of steel; f_s = the reinforcement steel stress, A_o = the area of concrete symmetric with reinforcing steel and can be considered as the total effective area of concrete in tensile zone of the girder divided by the number of longitudinal bars and c = concrete cover measured at the tensile face of the girder.

To compute maximum crack width at the extreme tension fibre of the cross section, multiply Equation (3) by factor given below;

$$\beta = \frac{D - d_n}{d - d_n} \quad (4)$$

D is the overall depth of the girder, d is the effective depth, measured from the top of the compressive face of the girder till the centroid of the tension bar and d_n is depth to the neutral axis, measured from the top of the compressive face.

3.2.4 Frosch

Crack width prediction formula proposed by Frosch [19] is based on the physical model of cracking and presented as:

$$w_c = \psi_s \sqrt{c^2 + \left(\frac{s}{2} \right)^2} \frac{f_s}{E_s} \quad (5)$$

w_c is the crack width at the level of reinforcement, s is the maximum longitudinal bar spacing and ψ_s is the crack spacing factor, through which, theoretical minimum, average and maximum crack widths can be computed using 1.0, 1.5 and 2.0 as multiplying factors, respectively. Authors use 2 as the crack spacing factor here.

To determine maximum crack width at the bottom of tensile surface of the girder, multiply Equation (5) by an amplification factor, as given in Equation (4).

3.2.5 Eurocode 2

To calculate maximum crack width as per Eurocode 2 [cited in 15], the difference between steel and concrete deformation has to be calculated. Formula used to calculate crack width is as follows:

$$w = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm}) \quad (6)$$

where, w is the crack width in a reinforced concrete member, $s_{r,max}$ is the maximum crack spacing, ε_{sm} is the mean strain in tensile steel at design loads and ε_{cm} is the mean strain in concrete between cracks.

To calculate final maximum cracking and difference in deformation between steel and concrete refer to equations given below;

$$s_{r,max} = 3.4c + 0.425 k_1 k_2 \frac{\phi}{\rho_{p,eff}} \quad (7)$$

where, k_1 is bond factor (0.8 for high bond bars, 1.6 for bars with plain surface, such as, prestressing tendons), k_2 accounts for strain distribution coefficient (1.0 for tension and 0.5 for bending) and $\rho_{p,eff}$ is effective reinforcement ratio including eventual prestressing steel.

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{f_s}{E_s} - k_t \frac{f_{ct,eff}}{E_s \rho_{p,eff}} (1 + \alpha_e \rho_{p,eff}) \quad (8)$$

k_t is the factor that depends on the duration of loading (0.6 for short term loading and 0.4 for long term loading) and α_e is the ratio of E_s / E_{cm} .

3.3 CASE STUDIES USED TO DESCRIBE THE STUDY

To discuss the usability of methodology and outputs extracted from above mentioned equations, it was essential to select appropriate and aging case studies.

Authors conducted several meetings with regional asset owners and noticed that majority of bridges constructed around 1950's and 1960's primarily used "U slabs" for construction purposes; also referred as girders in further discussion. These units acted as a girder as well as slab, where a considerable depth of fill is placed on top to assist in lateral load distribution between slabs. Initially, normal strength reinforced concrete, $f'c = 29.12$ MPa was used for construction purposes, until high strength reinforced concrete u-slabs were introduced in 1962. Spans of these slabs ranged between 1.22 metres to 10.5 meters. These slabs were designed to cater for either half of wheel load or 0.70 of wheel load, depending on the span. As Australian design codes have gone through three major revisions, between 1960 and 2010, these

slabs have been extensively used and subjected to heavy repetitive loading since the construction phase. With continuous growth of the nation, demand is expected to grow and to fulfil the demand it is necessary to increase the fleet sizes. Further increase in truck sizes, means increasing legal load limit, which might have severe influence on existing reinforced concrete U slabs. Hence, authors decided to quantitatively evaluate these slabs and provide a time dependent assessment tool to asset owners.

A typical section of a standard U slab is shown in Figure 2. Table 1 and 2 provide structural details and material properties of the slabs in use. Authors chose four typical spans to perform the analysis, as shown in Table 1.

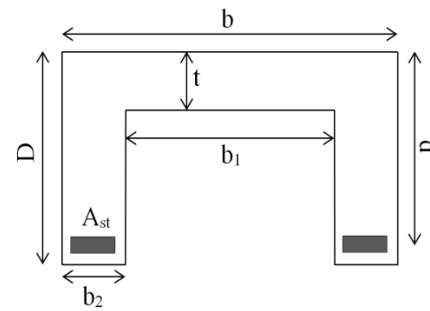


Figure 2: A typical U-Slab drawing used in Australia in 1950's and 1960's.

Table 1: Structural details of U-Slabs

Span		b	b ₁	b ₂	D	Bar Type
m	ft	(mm)	(mm)	(mm)	(mm)	
4.57	15	609.6	285.75	161.93	304.8	5 N 24
6.10	20		292.10	158.75	342.9	
7.62	25		298.45	155.58	393.8	6 N 24
9.14	30		304.80	152.40	444.5	7 N 24

Table 2: Material Properties of U-Slabs

Concrete Compressive Strength, $f'c$ (MPa)	29.12
Yield Strength of Steel, f_{sy} (MPa)	345
Elastic Modulus of Steel, E_s (MPa)	20,000
Cover (from the bottom edge of tensile zone to the closest bar), (mm)	40.875

3.4 ASSUMPTIONS

In this study, authors make certain assumptions to calculate load induced crack width. Cover of the girders is kept a constant value; no reduction in area of steel is accounted for due to possible corrosion. Design traffic loads specified in the subsequent code provisions in 1992 and 2010 [20, 12] since the girders' original design in early 1960s, are used as proof loads to assess the flexural stress levels and the resulting crack widths under serviceability limit state loads. To elaborate the

extreme stress values experienced by the U slab girders, well beyond the permissible stress limits, if subjected to 1992 and 2010 specified loads are reported in a linear relationship. Some studies [21] indicate up to 60% over strength is possible considering allowable stress design methods used in early 1960s, strain hardening of steel and concrete strength gain over long periods. However, the same crack width outcomes could be reached when an elastic perfectly plastic behaviour is assumed, beyond a desired permissible stress level determined by the assessor. Reported stress levels and crack widths are resulting from specified point axle loads and uniformly distributed loads specified in 1992 and 2010 revised traffic loads. As mentioned before, the reported results are based on serviceability limit state criteria.

4 RESULTS AND DISCUSSION

4.1 CRACK WIDTH VARIATION WITH CHANGE IN SPAN

Load induced crack widths are shown in Figures 3-6 for chosen spans. Crack widths range between 0.19 mm to 0.92 mm based on the applied proof loads. Figure 3 indicates the lowest value of crack width, i.e. 0.19 mm, whereas, maximum crack width, i.e. 0.92 mm is seen in Figure 4. Both values are for two different spans, where applied proof load is same.

Loads are applied at three stages since construction, i.e. 1960, 1992 and 2010. For 2010, stationary and moving traffic loads are considered. In order to compute maximum and average crack widths, truck loads are used to generate maximum bedding moment and steel stress at serviceability level. Crack widths are then drawn against steel stress because it is directly proportional to calculated crack width value, as per above mentioned equations, except concrete cover. It is quite evident from Figures 3-6, that, steel stress influences the growth of crack width with time. Concrete cover also accounts for a major influence on crack width computation, but variability of cover and its effects is outside the scope of this paper.

As per Figures 3-6, crack widths range between 0.19 mm to 0.31 mm in 1960, with steel stress varying from 236.94 MPa to 291.83 MPa, as per different models. At this stage, steel stress is in linear-elastic range and lower than yield limit, i.e. 350 MPa. Similar range of crack width is noticed at later time intervals, where it varies from 0.33 mm to 0.64 mm in 1992 with steel stress ranging between 417.87 MPa and 506.63 MPa. In 1992, steel stress generated due to proof load surpasses the yield strength of steel by $1.25f_{sy}$ to $1.5f_{sy}$, depending on the model considered. In 2010, theoretical steel stress value crosses the ultimate tensile strength zone, i.e. beyond $1.5f_{sy}$ and returns higher crack width values, such as, 0.38 mm to

0.82 mm for stationary traffic and 0.45 mm to 0.92 mm for moving traffic. Steel stress in 2010 varies between 485.15 MPa and 753.89 MPa for both traffic conditions. Increment in steel stress value beyond the yield point and tensile strength zone is one of the assumptions which authors consider to examine the quantitative behaviour of the girders. In addition, authors also suggest that a girder designed in 1960 might not see such traffic loading. Loads used here are placed in a manner that they generate high values. Above mentioned computations only indicate the influence of increase in truck loads with time, which are calculated by positioning the axle loads at different locations on effective spans.

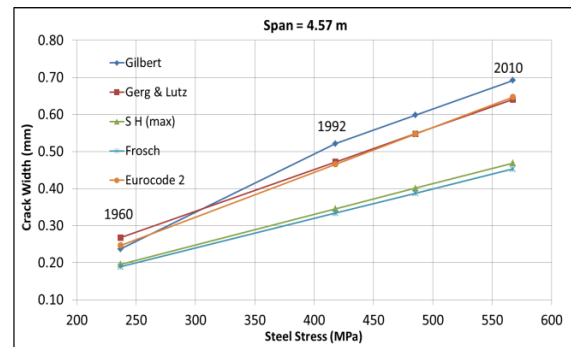


Figure 3: Crack width vs. steel stress for span of 4.57 m

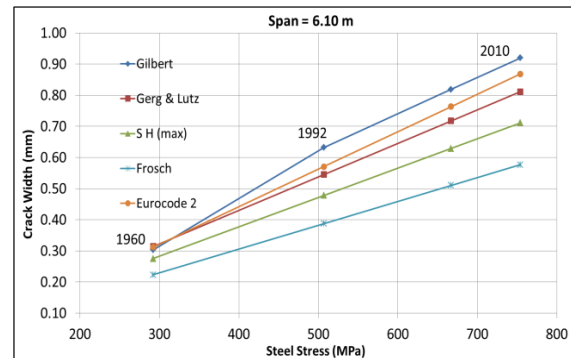


Figure 4: Crack width vs. steel stress for span of 6.10 m.

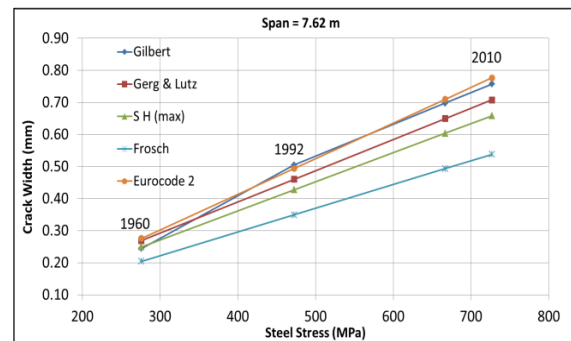


Figure 5: Crack width vs. steel stress for span of 7.62 m.

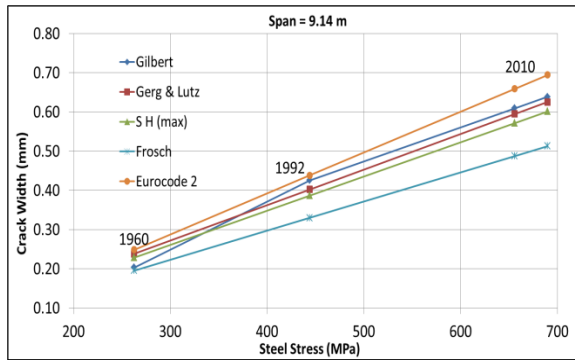


Figure 6: Crack width vs steel stress for span of 9.14 m

Models used to perform the analysis show variability in crack widths at every load interval, with a respective steel stress value. These crack widths further vary with change in span length. Such spectrum of crack widths at every load interval and span, provides a bandwidth with an upper and lower bound value, which can later be used for informed decision making by asset owners. Gilbert's model projects an upper bound value for spans 4.57 m and 6.10 m, whereas, Eurocode 2 estimates for 7.62 m and 9.14 m span lengths. On the other hand, Frosch's model projects lower bound value for all spans.

4.2 AGE OF INTERVENTION AND VALIDATION

To identify the right age of intervention and describe the application of methodology at asset management level, authors use detailed analysis of 9.14 m span as an example.

As per Figure 7, girder reaches 0.3 mm crack width, regarded as critical crack width, at different steel stress values considering several models. As per Euro code 2, girder reaches its critical value at 320 MPa and according to Frosch, approximately at 405 MPa. Both figures suggest different timelines between 1960 and 1992, when girder reaches its critical value.

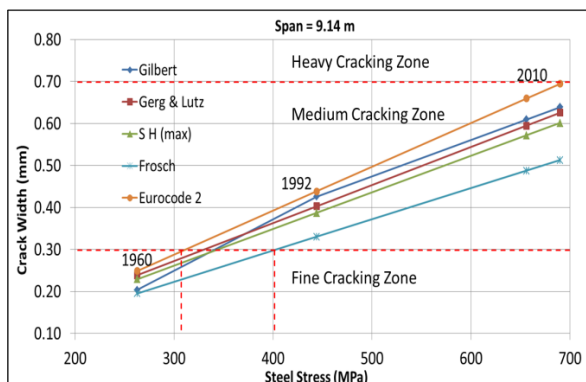


Figure 7: Critical crack width vs. steel stress at various ages for span = 9.14 m

As per VicRoads bridge inspection manual, 0.1 to 0.3 mm is considered as fine cracking, 0.3 to 0.7 mm is considered medium cracking and more than 0.7 mm is regarded as heavy cracking. These crack stages define visual condition state 2, 3 and 4 respectively, as per manual. But, recording fine cracking is a challenge and hard to conceive from naked eyes; hence remain undocumented in visual recordings. Also, measuring a crack from a far distance can result in an inaccurate recording. In a recent condition assessment report provided by the participating rural council, such undocumented recordings are noticed for above case study. As per the report, the girder's age is approximately 55 years and barely has any cracks; not even hairline which is less than 0.1 mm. Later, authors visited the site and inspected the girder in detail. Authors located some flexural cracks appearing to the soffit of the girder, as shown in Figure 8 and some flexural cracks in the web zone, as shown in Figure 9. Cracks are highlighted in red circles in Figures 8 and 9. From captured images it can be established that, cracks definitely exist but didn't get recorded. After comparing the visual cracks with crack size chart provided by VicRoads, it can be concluded that crack width ranges between 0.1 to 0.6 mm. That means the structure is in condition state 3 and prone to reach heavy cracking, if not maintained on time.



Figure 8: Cracks appearing at the soffit of the girder for 9.14 m span.



Figure 9: Flexural cracks appearing in web and at soffit the soffit of the girder.

Similar trend of crack widths is seen in Figure 7, based on quantitative assessment. As per analysis, crack width ranges between 0.48 mm to 0.70 mm in 2010, based on several models, which is identical to what is captured during site visits. Therefore, such evaluations can supplant and enhance the decision making, which are predominantly made based on the subjective data. Such methodology can be applied to any reinforced concrete girder to identify the crack widths with time, depending on the loads applied. Once, crack width reaches a critical value, i.e. 0.3 or 0.41 mm, asset owners can tag the bridge which might need detailed intervention in some years of time. For this example, authors can consider 2010 as the right time to intervene (TAoI) and inspect the structure in detail to stop any further degradation and corrosion setting. Proactive measures can be taken to restrict the crack growth, such as, applying the epoxy paint to the tension face.

Proposed methodology provides lead time to asset owners to take proactive decisions and identifying the quantitative status of the girder. Methodology is not only restricted to reinforced concrete girders, but can also be applied to pre and post tensioned girders after conducting some amendments in the present models.

5 CONCLUSIONS

Visual inspection is predominantly used to evaluate the aging bridges and recorded subjective condition data is used for future decision making. Authors propose a methodology to quantitatively assess the girders and support decision making. Load induced crack widths are calculated as quantitative performance measure. Several empirical models are used to perform the analysis and showed some promising results. Validity of the methodology is confirmed after comparing the quantitative analysis of 9.14m span with field observations. Asset owners can use this feasibility study as a handy, yet cost effective quantitative tool to take informed maintenance and monitoring decisions.

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