Assessment of Residual Strength and Residual Life of Concrete Beams

Mrs. Neethu Urs and Dr. M. N. Hegde
Associate Professor, Department of Civil Engineering,
Dayananda Sagar College of Engineering, Bengaluru, Karnataka, India.

Professor, Department of Civil Engineering,
Dr. Ambedkar Institute of Technology, Bengaluru, Karnataka, India.

Abstract
The structure is subjected to lot of stress during its lifetime. This in turn might lead to the damage of structures. Rehabilitation of infrastructure which are damaged due to earthquake load, fatigue load, overloading and environmental attacks during the service life is of great concern. Cracking in concrete reduces the load carrying capacity and accelerates the deterioration, which in turn shortens the service life of concrete structures. In order to refurbish the performance of the structure, the main requirements are to find the structure’s residual strength and residual life. This work is oriented towards the determination of residual strength and residual life of concrete beams. The beam is subjected to three point bending test. The results obtained experimentally are used to determine the principal stresses in different locations and also stress intensity factors using Finite Element analysis in a two dimensional medium. Using different failure theories, the obtained principal stresses are correlated to determine the residual stress and hence residual strength is determined. Using fatigue life measuring laws, the residual life is estimated. The residual life of the beam is measured in terms of number of cycles of load which it can carry safely without undergoing failure.

Keywords: Residual Strength, Residual Life, Stress Intensity Factor.

INTRODUCTION
Due to the technological advancements and changing user expectations, the asset owners are facing problems in decision making i.e., whether to refurbish the existing building and bring it back to its life or to completely demolish and rebuild. The option of demolishing and rebuilding have practical issues like decanting, access during construction, recycling of wastes and unforeseen costs besides many others. It is also noted that the refurbishment costs in many cases are approximately half the cost of new construction. With the expanding concentration on the supportability of the assembled environment, the choice of bringing back the serviceable life of structures (with least speculations) is discovering the best alternative amongst resource proprietors. In bringing back the serviceable life, one of the imperative contemplations is to find the Remaining Service Life (RSL) of the building in general or some of its segments. Urs, N., Manthesh, B. S., Jayaram, H., and Hegde, M. N. (2015) stated that, in nonexclusive terms, the remaining service life is an estimation, of the staying helpful administration of a building or part, making into note of its current condition and future working.

At many instances, during the service period of structure, one may be interested to estimate the remaining or residual service life of concrete elements as well as for the structure as a whole.

Figure 1: Indicative deterioration of structure with time

[CONTECVET-A validated users manual for assessing residual service life of concrete structures]

Murthy, A. R. C., Palani, G. S., Iyer, N. R., and Gopinath, S. (2012) concluded that Residual service life assessment requires obtaining first hand information regarding the current condition of the structure through a thorough condition survey. Such condition survey involves non-destructive and semi-destructive tests to obtain the strength and other properties of the concrete. Hovde, P.J. and Moser, K. (2004) suggested that ISO methods can be used to incorporate a probability distribution for the below mentioned factors and thus specify a distribution for estimated service life rather than deterministic estimates

\[ ESL = RSL \times f_A \times f_B \times f_C \times f_D \times f_E \times f_F \times f_G \]

Where,
ESL = Estimated Service Life; RSL = Reference Service Life; fA: Quality of Component; fB: Design Level, fC: Work Execution; fD: Indoor Environment; fE: Outdoor Environment; fF: In use Condition; fG: Maintenance.

The analysis of fatigue crack growth in concrete is complicated due to its heterogeneous nature. Thus, a statistical/probabilistic framework is needed for modeling of crack growth. Furthermore, a wide range of parameters may influence fatigue
Crack growth rates in concrete. These include mechanical as well as material parameters, such as fracture toughness, stress amplitude and stress range. A reliability assessment has been performed by Melchers, R.E. (1999) for fatigue crack propagation considering fracture toughness and the applied stress as primary random variables. The MEDIC [Méthode d’Évaluation de scénarios de Dégradation probables d’investissements Correspondants] method is based on a typical classification of a given element into four degradation schemes that quantify the past and future degradation behavior was proposed by Venkatesan et al (2006). The service life can be defined by the time to achieve a maximum acceptable probability of the serviceability of a limit state being reached as defined by Bamforth, P., (2003).

Comprehensive experimental program was conducted by Li et al (1999) aiming to investigate strength and serviceability deterioration of concrete structures, based on half-cell potential, depth of chloride penetration and chloride concentration. Fatigue performance of concrete member was evaluated empirically based on the well-known S-N curve approach by Verma et al (2013). Suitable numerical procedure for studying structural behavior of RC beams subjected to corrosion was developed by Coronelli and Gambarova, (2005) using nonlinear FEA. Based on the reduction of concrete porosity due to calcium carbonate, using nonlinear Finite Difference program, 2D coupled model was developed to analyze the deterioration of RC caused by chloride and carbonation, considering the effect of temperature and relative pore humidity. Puatatsananon and Saouma (2005) concluded that use of Artificial Neural Network provides more reliable result with the developed 2D model.

The numerical studies on three-point bending concrete specimens considering tension softening effect was conducted by Sain and Kishen (2008). The ultimate moment capacity was calculated based on equivalent strain concept. FE based model to evaluate the service life of RC Structures in three key steps, chemical ingress, steel corrosion and concrete cracking was developed by Pan and Wang (2011). To predict the corrosion initiation time, chloride penetration process in varying environment was evaluated by Cheung et al (2009). Micromclimatic variation on the concrete surface has also been investigated.

Song et al (2007) predicted the Service life of RC structures through micromechanics based corrosion model such as corrosion initiation period, corrosion propagation period, corrosion acceleration period and deterioration period. Mathematical model based on Fick’s second law of diffusion and other previous models was used by Liang et al (2003) to study the service life of RC bridges which had three stages, the initiation time, the depassivation time and the corrosion (propagation) time.

Computational methodology for predicting life cycle and estimating the service life of bridges through latest modeling tool namely incremental nonlinear FEA was proposed by Okasha and Frangopol (2010). Sain and Kishen (2008) concluded that multiple cracks can be represented as an equivalent single crack using damage index. The damage index, defined using the minimum eigen value of the stiffness matrices is independent of the size of the specimen for geometrically similar specimens.

Murthy et al (2012) predicted Residual strength using tension softening models and observed that the predicted residual strength is in good agreement with the corresponding analytical values in the literature. Based on the studies, it can be concluded that the predicted residual moment using modified bilinear model may be correct. Verman (2013) observed that service life of a structure has three major phases-time after construction and before corrosion initiation. Time between corrosion initiation and crack formation: time period after crack formation before failure of structure.

The present work focuses on the determination of residual strength using failure laws and residual life using Paris Law, Foreman Law and Walker Law.

### OBJECTIVES AND METHODOLOGY OF THE STUDY

Structures for example bridges, towers, buildings etc, which are continuously subjected to loads are important for the progress of society. Damages in structures can cause catastrophic effects. The safety and reliability of a structure is one of the key issues encountered in the design of a structure. A critical assessment of the damage and its effect on a structure carrying load is of prime importance. The objective of the present study is to find the residual strength and residual life of a damaged RC beam.

The beams with single edge notches are evaluated, when it fails. With the available results FEA tool ANSYS is used to model and analyze hence determine the three principle stresses at the crack tip and at other distant nodes. These stresses are used to evaluate the failure stress using failure theories. With the known failure stress due to applied load and the failure stresses obtained for different loading cases analysed at different nodal distance is compared to find the Residual Strength.

For the fatigue failure load the stress intensity factor is determined through FE analysis. The same is used in the fatigue life measuring laws such as Paris’ Law, Foreman Law and Walker Law to estimate the Residual Life.

### EXPERIMENTAL ANALYSIS

Single edge notched beam (SENB) of notch depth 20mm that is 20% of the thickness were prepared and tested for flexure by three point loading as suggested by Fathima and Kishen (2013). Failure load along with the corresponding deflection at regular interval of load were recorded.

**Figure 2:** SENB with initial crack and crack increment with load Fathima and Kishen (2013)

For the comparative study, M40 grade of concrete was designed as per IS 10262:2009 and the mix proportion is given below, C: Fa: Ca: W = 1: 1.76: 3.11: 0.5
FLEXURE TEST
Three point bending test was carried out on the specimens and the following test results are shown in the table below. The average failure load for the three specimens tested was found to be 5.6kN.

Table 1: Avg. Dial Guage and Crack Increment Readings

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Dial Guage(mm)</th>
<th>Crack Length(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>0.5</td>
<td>20</td>
</tr>
<tr>
<td>1.6</td>
<td>1</td>
<td>25</td>
</tr>
<tr>
<td>2.4</td>
<td>2</td>
<td>30</td>
</tr>
<tr>
<td>3.2</td>
<td>4</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>4.8</td>
<td>6</td>
<td>45</td>
</tr>
<tr>
<td>5.6</td>
<td>8</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 3: Experimental setup of SENB with three point bending

STIFFNESS
The stiffness ‘K’ of a specimen is a measure of the resistance offered by the specimen to an applied load. Table 1 shows the stiffness for specimen tested with respect to the load. Figure 4 shows that with the increment in load, the stiffness of the specimen reduces gradually. This plot of reduction of stiffness versus crack length is called as stiffness degradation curve. Stiffness is calculated by the following relationship,

\[ K = \frac{P}{\delta} \]

where, \( P \) = Load and \( \delta \) = Deflection.

Table 2: Stiffness of Specimen Tested

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Stiffness (N/mm)</th>
<th>Crack Length(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>1600</td>
<td>20</td>
</tr>
<tr>
<td>1.6</td>
<td>1600</td>
<td>25</td>
</tr>
<tr>
<td>2.4</td>
<td>1600</td>
<td>30</td>
</tr>
<tr>
<td>3.2</td>
<td>1280</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>40</td>
</tr>
<tr>
<td>4.8</td>
<td>872.7273</td>
<td>45</td>
</tr>
<tr>
<td>5.6</td>
<td>700</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 4: Stiffness Degradation Curve

COMPLIANCE
The inverse of stiffness is compliance C of a specimen. The Table shows the compliance for the specimen tested. From the graph obtained from compliance versus crack increment shows that with the increment in load the compliance of the specimen increases gradually. This compliance curve which gradually increases can be used to find out the critical crack length ‘a_c’. Compliance is given by the following relationship,

\[ C = \frac{\delta}{P} \]

where, \( P \) = Load and \( \delta \) = Deflection.

Table 3: Compliance of Specimens Tested

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Stiffness (mm/N)</th>
<th>Crack Length(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>0.000625</td>
<td>20</td>
</tr>
<tr>
<td>1.6</td>
<td>0.000625</td>
<td>25</td>
</tr>
<tr>
<td>2.4</td>
<td>0.000625</td>
<td>30</td>
</tr>
<tr>
<td>3.2</td>
<td>0.000781</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>0.001</td>
<td>40</td>
</tr>
<tr>
<td>4.8</td>
<td>0.001146</td>
<td>45</td>
</tr>
<tr>
<td>5.6</td>
<td>0.001429</td>
<td>100</td>
</tr>
</tbody>
</table>

From Figure 5 it can be seen that as the load increases compliance gradually increases. The minor grid lines along X and Y axis clearly shows that the critical length of the specimen is 45 mm.

Figure 5: Compliance Curve
FE ANALYSIS
The aim of this analysis is to determine the Principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$ for failure load of 5.6 kN obtained from the experimental analysis and also to determine the principal stresses for decrement loading with respect to failure load such as 5, 4, 3, 2 and 1 kN respectively, to determine the residual strength.

FINITE ELEMENT MODEL DEVELOPMENT OR PREPROCESSING
The finite element model was developed using isoparametric curved solid elements (SOLID186) in ANSYS. The material used for the present case study is Concrete. The Young’s Modulus and Poisson’s ratio are to be defined for the analysis. The Young’s modulus for concrete is taken as 31622.77 MPa. The Poisson’s ratio for concrete is 0.15. The boundary condition and load applied are similar to the experimental analysis.

SOLUTION
The FE analysis is carried out with prescribed boundary conditions and load. The following are the results obtained. The deformed model is shown in the Figure 8.

Figure 6: Finite Element Model of Single Edge Notched Beam

The crack tip is characterized by a set of special elements known as the singular elements. A total of 64 singularity elements are generated around each crack tip, thus maintaining the singularity element angle of 5.625° as shown in figure.

Figure 7: Enlarged View of Crack Tip

Figure 8: Deformed Model

Figure 9: 1st Principal Stress

Figure 10: 2nd Principal Stress

Figure 11: 3rd Principal Stress
For 5.6 kN load obtained from the experiment, the principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$ are shown in Fig 11. Since the principal stresses were very high at the crack tip, the quarter point nodal elements were chosen, six different nodal distances were taken from the crack tip and the corresponding principal stresses were determined.

Table 4 shows the principal stresses for the failure load of 5.6 kN at different distance from the crack tip.

<table>
<thead>
<tr>
<th>Distance(mm)</th>
<th>$\sigma_1$(MPa)</th>
<th>$\sigma_2$(MPa)</th>
<th>$\sigma_3$(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>17.648</td>
<td>11.627</td>
<td>1.447</td>
</tr>
<tr>
<td>2</td>
<td>6.6573</td>
<td>5.549</td>
<td>0.112</td>
</tr>
<tr>
<td>2.5</td>
<td>6.6354</td>
<td>5.483</td>
<td>0.1253</td>
</tr>
<tr>
<td>3</td>
<td>6.6134</td>
<td>5.417</td>
<td>0.1379</td>
</tr>
<tr>
<td>3.528</td>
<td>6.1318</td>
<td>5.022</td>
<td>0.1025</td>
</tr>
<tr>
<td>4.056</td>
<td>5.6501</td>
<td>4.6268</td>
<td>0.06721</td>
</tr>
</tbody>
</table>

Figure 12: Principal Stress Variation for 5.6 kN

Similar analysis was carried out for 5.0, 4.0, 3.0, 2.0 and 1.0 kN respectively. The following are the graphs for respective loadings.

Figure 13: Principal Stress Variation for 5.0 kN

Figure 14: Principal Stress Variation for 4.0 kN

Figure 15: Principal Stress Variation for 3.0 kN

Figure 16: Principal Stress Variation for 2.0 kN

Figure 17: Principal Stress Variation for 1.0 kN
With all the values of principal stresses, the assessment of residual strength is carried out in the following section.

**RESIDUAL STRENGTH**

The determination of residual strength for un-cracked structures is simple because the ultimate strength of the material is the residual strength. Crack in the structure causes high stress concentration resulting in the reduced residual strength. When the load on the structure exceeds a certain limit, the crack will extend. The crack extension may become immediately unstable and the crack may propagate in a fast uncontrollable manner causing complete fracture of the component.

**FAILURE CRITERION ASSUMED**

The following failure theories are considered:

- **Maximum principal stress theory (Rankine Theory):**
  \[
  \sigma_1 \geq \sigma_{ult}
  \]
  \(1\)

- **Maximum shear stress theory (Tresca Theory):**
  \[
  \frac{(\sigma_1 - \sigma_3)}{2} \geq \sigma_y/2
  \]
  \(2\)

- **Shear strain energy theory (Von Mises Theory):**
  \[
  \sqrt{\frac{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}{2}} \geq \sigma_y
  \]
  \(3\)

The ultimate stress was determined using the pure bending equation of a beam.

\[
\frac{M}{I} = \frac{\sigma}{y}
\]

Where,

- \(M\) = Moment due to load, N
- \(I\) = Moment of Inertia, mm\(^4\)
- \(\sigma\) = Ultimate Stress, MPa
- \(y\) = The distance of extreme layer of cross section from centroidal axis of beam, mm

**RESIDUAL STRENGTH EVALUATION**

Residual strength assessment for different nodal distances from crack tip with respect to failure theories are shown in Figure 18. Table 5 gives the stresses for different failure theories and also stresses due to experimental analysis for the nodal distance of 0.5mm from the crack tip.

Figure 18 indicates that with the increase in load at the crack tip, increases the stress intensity and hence the chances of failure is very high. The residual strength left in it is very negligible.

Hence in order to clearly understand, the stress intensity for various loads at different nodal distance the same analysis was carried out using FE method and hence residual strength is found out, Similarly graphs are plotted for the nodal distance of 2.0, 2.5, 3.0, 3.5, 28 and 4.05mm.

Table 5: Residual Strength for 0.5mm Nodal Distance.

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>(\sigma_1) (MPa)</th>
<th>(\sigma_2) (MPa)</th>
<th>(\sigma_3) (MPa)</th>
<th>(\sigma_1) Residual (MPa)</th>
<th>(\sigma_2) Residual (MPa)</th>
<th>(\sigma_3) Residual (MPa)</th>
<th>(\sigma_y) (MPa)</th>
<th>(\sigma_{ult}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.6</td>
<td>17.55</td>
<td>4.2</td>
<td>-15.34</td>
<td>8.10</td>
<td>2.1</td>
<td>-6.90</td>
<td>14.18</td>
<td>4.2</td>
</tr>
<tr>
<td>5.0</td>
<td>15.75</td>
<td>4.2</td>
<td>-11.55</td>
<td>7.23</td>
<td>2.1</td>
<td>-5.13</td>
<td>12.66</td>
<td>4.2</td>
</tr>
<tr>
<td>4.0</td>
<td>12.60</td>
<td>4.2</td>
<td>-8.40</td>
<td>5.78</td>
<td>2.1</td>
<td>-3.68</td>
<td>10.13</td>
<td>4.2</td>
</tr>
<tr>
<td>3.0</td>
<td>9.45</td>
<td>4.2</td>
<td>-5.25</td>
<td>4.33</td>
<td>2.1</td>
<td>-2.23</td>
<td>7.59</td>
<td>4.2</td>
</tr>
<tr>
<td>2.0</td>
<td>6.30</td>
<td>4.2</td>
<td>-2.10</td>
<td>2.89</td>
<td>2.1</td>
<td>-0.79</td>
<td>5.06</td>
<td>4.2</td>
</tr>
<tr>
<td>1.0</td>
<td>3.15</td>
<td>4.2</td>
<td>1.04</td>
<td>1.44</td>
<td>2.1</td>
<td>0.65</td>
<td>2.33</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Figure 19: Residual Strength at Nodal Distance of 2.0mm

From Figure 19 it can be inferred that up to 3kN, load the stresses are not exceeding the failure stress. Hence the structure can bear the static service load up to 3kN without failure. It is observed that the stresses at distant nodes from the crack tip is comparatively less than the crack tip.

Figure 20: Residual Strength at Nodal Distance of 2.5mm
With the increased nodal distance the stresses are comparatively low, Fig 20 shows that the specimen can take up to 4kN load as the difference between the failure stress due von-Mises theory is almost the same as that of the failure stress due to static load obtained from the experiment. It is noticed that the failure stresses obtained from the three different failure theories such as Rankine theory, Tresca theory and von-Mises theory are proportionate to each other.

![Figure 21: Residual Strength at Nodal Distance of 3.0mm](image)

From Figure 23 it can be observed that the principal stresses decrease with the decreasing load as well as with the increasing nodal distance from the crack tip. The decrease in the principal stresses from the crack tip to distant nodal point is noteworthy. The principal stress at 0.5mm nodal distance from the crack tip is 17.648 MPa for the failure load of 5.6 kN, whereas for the same failure load the principal stress at 2.0mm nodal distance is 6.657 MPa. From Figures 18-23 it is clear that the reduction in principal stress is about one third of the stress at the nodal distance close to crack tip.

At the nodal distance of 4.056mm from the crack tip, it can be derived that 4.0 kN is the permissible static service load. The values of failure stress obtained from three failure theories namely Rankine theory, Tresca theory and von-Mises theory are closer. Failure stress evaluation from von-Mises theory is a combination of all the three principal stresses whereas from Tresca theory it is a combination of first and third principal stress. In the case of Rankine theory it is just the first principal stress. Though all these theories hold good for brittle materials it is better to find the residual strength or residual stress of the concrete beams by using the von-Mises theory as it gives the failure stress corresponding to all the three principal stresses acting at that particular node.

### RESIDUAL LIFE
Crack growth is a result of cyclic loading or of the combined action of sustained loading and environment or both. The most common crack growth mechanisms are fatigue crack growth and environment-assisted (corrosion) fatigue crack growth. Fatigue cracking is difficult to prevent, but it can be controlled. To predict crack growth behaviour stress-intensity factor, described as a function of crack size, for the relevant structural and crack geometry is important parameter.

In order to find the residual life, the fatigue failure load is 30% of failure load is taken [14]. The fatigue failure load was found to be 1.68kN. For this fatigue failure load, the stress intensity factor along the crack front is determined using FEA.

#### Table 6: Stress Intensity Factor along Crack Front for 1.68kN

<table>
<thead>
<tr>
<th>Crack front (mm)</th>
<th>K_I MPa/√m</th>
<th>K_II MPa/√m</th>
<th>K_III MPa/√m</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.04696</td>
<td>0.01667</td>
<td>0.077343</td>
</tr>
<tr>
<td>25</td>
<td>0.04658</td>
<td>0.00834</td>
<td>0.0806</td>
</tr>
<tr>
<td>50</td>
<td>0.03195</td>
<td>0.001154</td>
<td>0.000518</td>
</tr>
<tr>
<td>75</td>
<td>0.5892</td>
<td>0.000845</td>
<td>0.864</td>
</tr>
<tr>
<td>100</td>
<td>0.0798</td>
<td>0.00263</td>
<td>0.0922</td>
</tr>
</tbody>
</table>

From the above stress intensity factor using Tanaka Model the equivalent Stress intensity factor is determined.

**Tanaka Model:**

\[
\Delta K_{\text{eqv}} = (\Delta K_I^4 + 8\Delta K_{II}^4 + 8\Delta K_{III}^4(1-\nu))^{25}
\]

Where,

- \( K_I \) = Stress Intensity Factor for Mode 1 failure.
- \( K_{II} \) = Stress Intensity Factor for Mode 2 failure.
- \( K_{III} \) = Stress Intensity Factor for Mode 3 failure.
- \( \nu \) = Poisson’s Ratio of material.

Table 7 shows that, at the crack front of 75mm the equivalent stress intensity factor is high compared to the other crack fronts.
Hence for the determination of residual life using the fatigue life measuring laws this value is used.

**Table 7: Equivalent Stress Intensity Factor along Crack Front for 1.68kN**

<table>
<thead>
<tr>
<th>Crack Front (mm)</th>
<th>$K_{eqv} MPa\sqrt{mm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.136016</td>
</tr>
<tr>
<td>25</td>
<td>0.141593</td>
</tr>
<tr>
<td>50</td>
<td>0.03195</td>
</tr>
<tr>
<td>75</td>
<td>1.521942</td>
</tr>
<tr>
<td>100</td>
<td>0.163846</td>
</tr>
</tbody>
</table>

The below mentioned are the few parameters used in fatigue life measuring laws and it is taken with respect to the old literature available [14]. ‘C’ and ‘m’ are the Paris law coefficients and $K_{IC}$ is fracture toughness. The following are the values for a medium beam casted, $C = 5 \times 10^{-4}$, $m = 3.9$

$$K_{IC} = 30 \, MPa\sqrt{mm}$$

**RESIDUAL LIFE ASSESSMENT**

Paris’ Law is the most commonly used law for fatigue life analysis introduced by Paris and Erdogan (1961). The model is simple in use and only needs two curve fitting parameters to be determined.

$$\frac{da}{dN} = C(\Delta K)^m$$

$$\frac{dN}{da} = 5 \times 10^{-4} \times 1.521942^{3.9}$$

$$\int dN = \int_{20}^{80} \frac{da}{5 \times 10^{-4}(a) + 1.521942^{3.9}}$$

$N = 23327.88$ cycles

‘N’ indicates the estimation of residual life in terms of number cycles of load the particular beam can carry without undergoing failure. Hence the structure can be evaluated with respect to daily service load it carries and the residual life can be estimated in terms of time.

**CONCLUSIONS**

This paper presents a methodology to evaluate the residual strength using the failure theories and assessment of residual life using fatigue life measuring law, i.e. Paris Law. Residual strength is evaluated for the case of static failure load and that for residual life, fatigue failure load is incorporated. The results obtained indicates that the three failure theories used give proportionate failure stresses, hence all these theories hold good for the brittle failure. Since the von-Mises failure theory evaluates the failure stress with the combination of all three principal stresses hence residual strength evaluation can be done using von-Mises failure criterion.

**ACKNOWLEDGMENT**

First author like to thank Dr. J M Chandra Kishen who supported and guided to successfully complete the work and also Mr. Manthesh and Mr. Ajay Krishna V S who stood as a backbone until the end of this study with their suggestions and support.

**REFERENCES**

Diffusion in Concrete.” (June), 264-275. DOI: 10.1061/(ASCE)0899-1561(2005)17:3(264)


