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Mechanistic-empirical design of fibre reinforced concrete (FRC) pavements using inelastic analysis

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Abstract. Use of fibre reinforced concrete (FRC) for pavements is advocated since the higher crack resistance could lead to lower slab thickness and higher joint spacing. The post-cracking capacity of FRC allows pavements to be designed and analysed considering the response beyond the elastic regime. The current paper presents possible failure patterns in FRC pavement slabs, which are governed by the slab dimensions, loading type and boundary conditions, and the appropriateness of inelastic design methodologies for these failure patterns. Subsequently, a mechanistic-empirical design methodology developed for FRC pavements, based on yield line analysis incorporating fatigue in the moment calculation, is discussed. The proposed design methodology gives specific checks for the different failure patterns and the consequent design strategy to be adopted. The method incorporates material parameters, such as the first crack and post crack flexural strengths, and fatigue correction factors for the evaluation of the moment carrying capacity. Cumulative fatigue damage analysis is also done as a serviceability check. The final design solution satisfies both the inelastic moment capacity requirement and fatigue life required without excessive damage accumulation.

Keywords. Fibre reinforced concrete; rigid pavement; fatigue; yield line analysis; equivalent flexural strength.

1. Introduction

A carefully designed and constructed rigid pavement system could ensure long and maintenance free life. To this effect, recent design approaches adopt performance-based models intended to ensure optimized solutions with longer service life [1, 2]. Since such methodologies extend failure criteria beyond the elastic regime of concrete, crack formation and ductility have to be considered, especially under fatigue loading conditions. In this context, the characteristics of fibre reinforced concrete (FRC) make it suitable for improved performance of rigid pavements since the fibres enhance the crack resistance and rotation capacity of the concrete matrix significantly. Along these lines, the FHWA design method for ultra-thin white toppings (UTWT) provides for the use of fibres in overlay concrete [2]. Further, full depth FRC pavements have also been advocated by several researchers and practitioners [2–5]. Such pavements are more advantageous when inelastic design techniques are used that could result in the reduction of slab thickness, in comparison to plain concrete (PCC) pavements designed for the same stress levels [6–9]. Further, the incorporation of fibres extends the fatigue life of concrete significantly, which is of primary importance in pavements [10–17].

In India, the design of FRC pavements is guided by IRC SP:46-2013 [5], which uses the ultimate load design based on the circular yield-line pattern incorporating the equations suggested by Meyerhof for infinite slabs-on-grade [5, 18]. A major drawback of this method is that it assumes the slab to be of infinite dimensions, so as to allow the development of the complete circular yield line pattern. The fixed edge boundary conditions, perfect load transfer at joints and the absence of curling are other major conditions that are implicitly imposed due to the infinite slab assumption [2]. These assumptions limit the applicability of the design approach and necessitate the updation of IRC SP:46.

The present work extends the IRC method of FRC pavement design to incorporate responses arising from a range of combinations of load and boundary conditions. Further, the methodology incorporates material fatigue models in the design moment calculations.

2. Crack patterns possible in pavements

In general, the pavement slab is expected to be always in contact with the subgrade for the structural system to be most efficient [19–21]. Such a condition is not met when curling occurs, either upward or downward, leading to

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cracking when the wheel passes over the slab and the flexural strength is exceeded [19, 22–25]. Consequently, when there is such loss of contact between the slab and the ground due to curling, there is not much benefit in using FRC as the design is governed by the elastic limit or first-cracking, though it would reduce crack propagation due to shrinkage and other causes [26, 27]. There are several recommendations with respect to material selection, mix design and construction practices, in addition to design considerations, for mitigating curling [25, 28]. In terms of the panel dimensions, it is recommended that the largest joint spacing should be restricted to 4.5–6 m depending upon the slab thickness (*h*), with the smaller value for higher thickness [21, 29–31]; accordingly, a maximum joint spacing of $L = 24\,h$ was suggested by Delatte (2014) to ensure that no uplift occurs.

When curling does not occur, failure can occur with different crack patterns corresponding to combinations of loading configuration, dimensional and boundary conditions. The different cases that could occur are discussed below.

2.1 *D-cracking of slab*

D-cracking due to loading on the edge of the panel may occur if the width of the slab *(b)* is large enough and the uplift of the edges is completely prevented (refer figure 1). The minimum width required for this failure condition is related to the moment carrying capacity of the slab, the load applied and other parameters, an estimate of which has been derived by Ghosh and Dinakaran [19], as:

$$b \geq \left[\frac{P}{\pi(2l - {}^{c}\!/_{3})}\left(M_p - 0.1\alpha\Delta TEh^2\right)\right] \tag{1}$$

where M_p is the plastic moment capacity of slab, l is the radius of relative stiffness, c is the contact radius of the load P, α is the coefficient of thermal expansion of the concrete, ΔT is the maximum expected differential temperature in the slab and E is the elastic modulus of concrete. Here, yield lines develop at the bottom of the slab and the collapse condition can be defined as cracking at the top of slab. The limiting moment capacity would then be the sum of the plastic (positive) moment carrying capacity along the radial yield lines at the bottom and the elastic (negative) moment carrying capacity (at first crack) occurring along the circumferential yield lines at the top [3, 4, 29, 32, 33].

2.2 *Cracking due to transverse yield lines*

A transverse crack across the slab caused by the formation of yield lines, as shown in figure 2, could occur if the slab is narrow or has much smaller width in comparison to its length [19]; that is, b is less than the value obtained from Eq. (1). In this case, the limiting moment is governed by the plastic moment capacity as the slabs rotates about the two boundary cracks simultaneously. Consequently, the use of inelastic design method similar to the D-cracking case is possible for this failure pattern.

2.3 *Corner cracking*

This failure mode is initiated for cases where the load transfer mechanism at the corners is not designed with proper detailing so that the corner fails under load by cantilever action [2, 5]. The limiting moment will be governed by the elastic moment capacity as the cracks initiate at the top of the slab.

Keeping in view these failure mechanisms that could limit the pavement performance, a design methodology has been developed for the inelastic design incorporating fatigue characteristics.

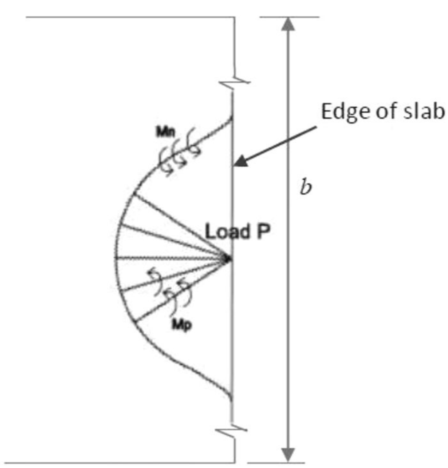

Figure 1. Cracking due to edge loads.

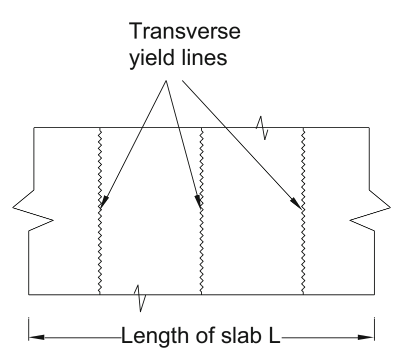

Figure 2. Cracking that occurs in narrow slabs.

3. Procedure for the design of FRC pavements

The inelastic design is performed here using yield line analysis for the most probable failure mechanism [3, 5, 18, 34, 35]. Accordingly, appropriate material parameters are to be employed for determining the moment carrying capacity along the yield lines. In FRC slabs, the fibre action is normally restricted to crack bridging and subsequently the concrete undergoes strain softening response [36, 37], which determines the moment carrying capacity along the cracks in the slabs [29, 30].

An estimate of the post-cracking strength of FRC can be obtained from flexural toughness tests, as suggested in IRC SP: 46-2013 [5]. The load deflection (P–δ) curve is obtained from flexural testing and the equivalent flexural strength, $f_{e,n}$, is calculated, as specified in JSCE SF4 (1984), IRC SP:46 (2013) or ICI-TC/01.1 (2014) [38–41]. The plastic moment capacity per unit length of the slab is estimated as:

$$M_P = f_{e,nk} \frac{h^2}{6} \tag{2}$$

where, $f_{e,nk}$ is the characteristic equivalent flexural strength of concrete and h is the thickness of the slab. The elastic moment capacity will be a function of the flexural strength (or modulus of rupture) of the concrete and is estimated as:

$$M_n = f_{ct,k} \frac{h^2}{6} \tag{3}$$

where, $f_{ct,k}$ is the characteristic value of the flexural strength of the concrete.

In the following sections, the step-by-step procedure for the design, including the dimension selection, adequacy check, fatigue check and final design solution, is presented.

3.1 *Step 1: Input parameters*

The required input parameters include traffic data, site layout, subgrade properties, climate and temperature data (if available), and any other data that could be relevant to the design such as tyre pressure.

3.2 *Step 2: Design assumptions*

Based on the expected load and site conditions, a minimum grade of concrete is to be considered, say M35. From the specifications regarding the load configuration, layout of the site, subgrade properties and related suggestions made in the section on failure patterns, the dimensions of the slab should be chosen such that there is no curling.

3.3 *Step 3: Estimation of material and other design characteristics*

The estimation of various design parameters including material characteristics, using appropriate relations taken from relevant codes and standards or from existing database, is done in the next step of the design. The major parameters are presented in table 1, along with the approach for their estimation.

The slab–subgrade interaction is represented by the radius of relative stiffness, based on the assumption of Winkler foundation, and is adopted as a conservative estimate. However, more accurate models representing the FRC slab–subgrade interactions may be adopted, if available.

3.4 *Step 4: Estimation of failure patterns*

Once the dimensions of the slabs are assumed (i.e., from Steps 2 and 3), the possible failure patterns can be estimated based on the joint spacing and width of slab, and the appropriate yield line mechanism is contemplated by checking for $L > 24\ h$ and for limiting b value from Eq. (1) [3, 5, 9, 18, 29, 30, 32, 42].

3.5 *Step 5: Estimation of plastic and elastic moment carrying capacities*

The plastic and elastic moment capacities are calculated as per Eqs. (2) and (3), and subsequently for the expected collapse mechanism, the allowable limiting moment is estimated using the appropriate expression given in Eq. (4), which is a modified form of design equations suggested in TR 34: 2003:

$$\begin{aligned} &M_{all} = (M_n + M_P) = \left(f_{ctk} + f_{en,k}\right)\frac{h^2}{6} \\ &\quad \text{(for circular/D - cracking)} \\ &\text{or} \\ &M_{all} = M_P = \left(f_{en,k}\right)\frac{h^2}{6} \\ &\quad \text{(for transverse cracking/corner cracking)} \end{aligned} \tag{4}$$

In case there are substantial flexural stresses due to thermal or shrinkage effects, they can be accounted for by reducing the elastic moment carrying capacity, along the same lines as suggested for the case of FRC slabs-on-grade design in TR 34 (2003, 2013). In order to incorporate this in the moment capacity estimate, corresponding flexural stresses due to temperature differential, $f_{\Delta T}$ and shrinkage f_{sh}, can be reduced from the total hogging moment capacity, as given in Eq. (5):

$$M_{all} = \left[f_{e,nk} + \left(f_{ctk} - f_{\Delta T} - f_{sh}\right)\right]\frac{h^2}{6} \tag{5}$$

3.6 *Step 6: Estimation of the modified moment capacity accounting for fatigue effects*

It is well established that the section capacity will be influenced by fatigue loading. Most design methods

Table 1. Details of design parameters to be estimated.

Parameter	Estimation method
Characteristic flexural strength of FRC, f_{ctk}	Experimental data (or) material database
Poisson's ratio, μ*	Standards/codes*
Elastic modulus, E*	Experimental data (or) from standards/codes*
Characteristic equivalent flexural strength of FRC, $f_{e,nk}$	Experimental data (or) material database
Contact radius, c	From tyre pressure and axle load data, as $c = \sqrt{\frac{P}{\tau\pi}}$ where, P is the load transmitted by a tyre, τ is tyre pressure.
Radius of relative stiffness, l	$l = \left[\frac{Eh^3}{12(1-\mu^2)k}\right]^{1/4}$, where k is the modulus of subgrade Note: If k is not available, it can be estimated using the CBR value from appropriate standards and codes.

*For the commonly used dosages of fibres, these material parameters maybe estimated using the expressions used for PCC.

account for this response by using dynamic load factors resulting in a conservative design solution. Such an approach fails to include the enhanced fatigue response due to the presence of fibres in concrete. Consequently, an approach wherein the material strength characteristics are suitably altered to represent the flexural response under repeated loading is more suited. In the present approach, reduction factors are applied to the strength parameters, used in the moment capacity estimates, to account for the loss in capacity due to fatigue. These reduction factors are to be obtained from fatigue models corresponding to the FRC mix used [16, 17, 43]. Equation (5) is then modified for the moment carrying capacity under fatigue loading, as given in Eq. (6):

$$M_{all} = \left[Y\frac{f_{e,nk}}{\gamma_c} + (X\frac{f_{ctk}}{\gamma_c} - f_{\Delta T} - f_{sh})\right]\frac{h^2}{6} \qquad (6)$$

where, X and Y are the reduction factors for the elastic moment carrying capacity and the plastic moment carrying capacity, respectively. The method for obtaining reduction factors is explained in the following section.

3.6a *Determination of strength reduction factors* For each load class, as given in the input data of the axle load spectrum, the corresponding expected maximum number of load repetitions, N has to be used to determine the reduction factor from the material fatigue model [9]. Appropriate fatigue models based on the stress ratio-fatigue life curves (S–N curves), have to be chosen for both uncracked (related to f_{ctk}) and post-cracked stages (related to $f_{e,nk}$) of the FRC, and the safe stress ratios (or X and Y values) corresponding to the assumed N-value are to be determined. This is illustrated in figure 3 where representative S–N curves of an M35 FRC with steel fibre volume fraction of 0.5%, for the uncracked and pre-cracked concrete, are used to obtain the reduction factors. For example, for a required $N = 1000$ (i.e., log $N = 3$), the safe stress ratio limits for FRC are given by $X = 0.92$ from Curve 1 (uncracked state), and $Y = 0.84$ from Curve 2 (cracked state).

3.7 *Step 7: Estimation of allowable collapse load*

From yield line analysis, using the expressions corresponding to each failure condition [18, 29, 30], the allowable load corresponding to the maximum allowable moment (obtained in Step 5) is calculated as:

$$P_{all} = M_{all}\, f(c/l) \qquad (7)$$

where c is the contact radius of the load and l is the radius of relative stiffness (see table 1) and the function $f(c/l)$ is based on the yield pattern and may be obtained from any suitable analysis, such as Meyerhof's analysis given in TR34 (2003, 2013), for circular/D-cracking.

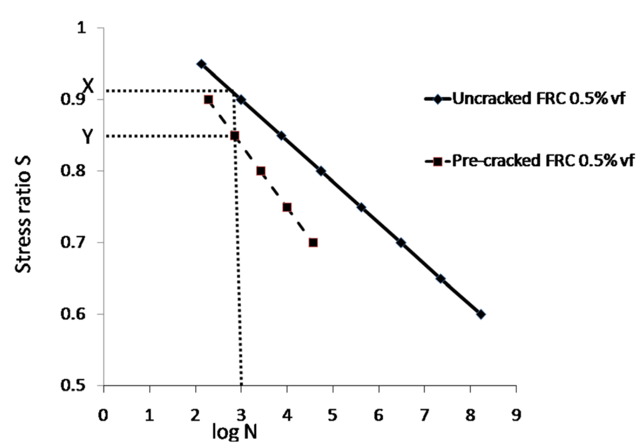

Figure 3. Example S–N curves for uncracked and cracked FRC.

Table 2. Results of the parametric study of the design methodology.

Dosage of steel fibres in kg/m^3	Subgrade modulus, k in N/mm^3	Required minimum thickness, h in mm	Predicted mode of failure as per the design
10	0.08	290	D/Edge cracking
10	0.14	290	D/Edge cracking
10	0.225	290	D/Edge cracking
20	0.08	290	Fatigue cracking
20	0.14	280	Both D/Cracking and Fatigue cracking
20	0.225	270	Both D/Cracking and Fatigue cracking
30	0.08	285	Fatigue cracking
30	0.14	275	Fatigue cracking
30	0.225	265	Fatigue cracking

3.8 *Step 8: Design Check 1- Allowable load > Maximum axle load*

If the allowable load is higher than the maximum applied load, then the combination of thickness, grade of concrete and dosage of fibres is sufficient with respect to the inelastic design. Otherwise, the designer has the choice to redesign using a higher pavement thickness, changing the grade of concrete or including fibres and dosages, which would give higher values for the material characteristics.

3.9 *Step 9: Fatigue damage check*

Since a separate serviceability check for fatigue has to be performed, the fatigue damage analysis is to be done. The analysis includes determining the cumulative fatigue damage (CFD) and applying Palmgren-Miner's rule to check for fatigue failure [2, 5, 44]. In order to perform a CFD analysis, it is essential to determine the stress ratio due to the applied load for estimating the allowable number of repetitions, as:

$$SR = \frac{stress\ \ due\ \ to\ applied\ \ load, \sigma_{app}}{allowable\ \ stress, \sigma_{all}} \tag{8}$$

It is recommended that the stress due to applied load should be calculated from the elastic analysis. In India, IRC 58 is a commonly adopted standard for rigid pavement design based on elastic stress analysis, and it is suggested that, in the absence of other solutions, the equations provided in Section 5.3 of IRC 58:2010 be used. However, the possibility of developing stress estimates based on inelastic analysis is being explored by the authors.

The allowable stress is related to the material parameter that would be representative of the moment carrying capacity. Consequently, the stress ratio expression can be given as:

$$SR = \frac{\sigma_{app}}{f_{ctk}} \tag{9}$$

From the *SR* obtained for each load class, the allowable number of repetitions may be evaluated using the appropriate *S–N* relation representing the fatigue response of FRC in the uncracked state, corresponding to the *S–N* curve used to determine the strength reduction factor *X*. Finally, Palmgren-Miner's rule should be applied to the cumulative value from all the load classes (refer Nayar and Gettu [9]). From the allowable and actual expected number of load repetitions for each load class (i), the CFD is obtained as:

$$CFD = \frac{\sum n_i}{\sum N_i} \tag{10}$$

As per Palmgren-Miner's rule, the design is safe if CFD < 1.

Thus the design accounts for safety with respect to both cracking due to inelastic stresses and fatigue damage through the two design checks.

3.10 *Parametric study*

A parametric study was done in order to understand the design solutions with respect to various variables. For the study, the minimum thickness required for a specific axle load spectrum was determined by varying the subgrade modulus and using FRC with various dosage of the same type of fibres. The designs were done considering D-cracking as the failure pattern. The results are reported in table 2, along with the expected failure pattern or mode.

As discussed earlier, there are two design checks: the first check for the maximum allowable load as per the inelastic moment capacity and the second check for the serviceability based on fatigue damage. The predicted failure mode obtained in the designs represents the condition when either one or both of the design checks become the governing criterion. It can be seen that for FRC with a low dosage of fibres (10 kg/m^3), where the post-cracking flexural strength is low, the slab possesses limited rotation capacity for the yield lines to develop, causing the slab to crack before fatigue cracking can occur. On the other hand, at higher fibre dosages, the design is essentially governed by fatigue failure; the higher flexural toughness renders the design to be safe with respect to the inelastic design moment (as seen in table 2) even at lower thicknesses but a higher slab thickness is required to satisfy the CFD check for fatigue damage based on the elastic stress.

4. Conclusions

The paper presents a design methodology for full depth FRC pavements. The methodology is proposed considering various failure conditions and criteria. The major conclusions are:

- The suggested design methodology for FRC pavements addresses all possible failure conditions, based on the dimensions/boundary conditions and curling response in the slab, and recommends either the elastic or inelastic design strategy for each case.
- For the inelastic design, the limiting moment is obtained assuming the failure condition to be the initiation of cracking at the top of the slab, and is a function of elastic and/or plastic moment carrying capacity of the slab.
- The plastic moment capacity is obtained from the post cracking flexural strength (equivalent flexural strength) of FRC and the elastic moment carrying capacity from the first crack flexural strength.
- Fatigue failure criteria are incorporated in the design by assigning reduction factors for the flexural and equivalent flexural strengths in the calculation of the section capacity.
- The parametric study illustrates that the design accounts for the critical failure mode (inelastic cracking or fatigue cracking) due to the two-level check on inelastic moment capacity and cumulative fatigue damage.
- The design solutions indicate that use of very low dosage (like 10 kg/m^3) results in the failure of slab under much lower stresses by flexural cracking before fatigue cracking occurs.

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