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Influence of axle load on the fatigue life of thin whitetopping with fibre-reinforced concrete

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Abstract. Whitetopping technique for rehabilitation of bituminous pavements has been considered as a cost-effective solution when the life cycle cost of pavement is considered. Thin whitetopping is a preferred overlay when both the structural and functional capacities of the pavement are required to be enhanced. IRC:SP:76-2015 outlines the procedure for construction of thin whitetopping in India. While IRC:SP:76-2015 suggests that thin whitetopping can be used on rural roads and medium to moderately heavy volume roads, the traffic composition related to the same is not clearly specified. In this study, an attempt was made to specify the limiting traffic characteristics for thin whitetoppings in terms of the axle load and their number of repetitions. To estimate the effect of traffic, the actual traffic data collected from four National Highways in India were used and the whitetopping design was carried out as per IRC:SP:76-2015. To increase the total load carrying capacity of the whitetopping overlay, especially in locations experiencing heavier traffic loads, mixes with different dosages of polypropylene fibres were designed and silica fume was added to enhance the bond between fibres and concrete. Eight different mixes were prepared with different combinations of fibre dosage (0.8%, 1.3% and 1.8%) and silica fume (8% and 13%) in addition to a control mix and a mix with 8% silica fume and no fibres. The mix with 8% silica fume and 1.8% fibres exhibited the highest equivalent flexural strength and the control mix with no fibres and silica fume exhibited the lowest equivalent flexural strength. These two mixes were considered to estimate the effect of axle loads. In most cases, it was seen that thin whitetopping without fibres failed in design for the actual traffic conditions as the fatigue response was highly sensitive to the axle loads. When the axle loads were limited closer to the legal axle loads specified in IRC:3-1983, it was seen that even the mix without fibres was able to sustain the design traffic.

Keywords. Whitetopping; fibre-reinforced concrete; residual flexural strength; axle load.

1. Introduction

In many countries across the world, pavements undergo premature failure much before their stipulated design life. While structural failures are commonly observed in aged bituminous pavements, most of the premature failures observed in relatively new bituminous pavements are related to their functional aspects. To restore the functional aspects of such bituminous pavements, a number of overlay options are currently available. On a broader scale, the overlay options could be classified as a bituminous overlay or a concrete overlay. While a bituminous overlay is generally preferred, a number of studies have shown that a concrete overlay, commonly called as whitetopping, could be cost competitive when the life cycle cost of the pavement is considered [1–3]. When rutting or surface cracking is the main distress in a bituminous pavement, adopting whitetopping is considered to be an effective and sustainable technique for rehabilitation [4, 5].

The whitetopping is further categorised as conventional/thin/ultrathin depending on the thickness of the overlay and the degree of bonding between the concrete overlay and the existing bituminous pavement [6, 7]. The whitetopping overlay is categorised as thin when the thickness varies between 100 and 200 mm and ultrathin when the thickness varies between 50 and 100 mm. In the case of ultrathin whitetopping (UTWT), a bond between the overlaid Portland Cement Concrete (PCC) and underlying bituminous layer is considered in the design and appropriate measures are taken to ensure that such a bond is created during construction. However, for thin whitetopping (TWT), the bond between the overlaid PCC and underlying bituminous layer is often a consideration but it is not mandatory. Hence the bonding consideration is ignored in the design.

UTWT contributes scarcely to the strength aspects of the pavement and it typically upgrades the pavement from fair condition to excellent condition [6]. TWT enhances the

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structural capacity of the pavement to a certain degree in addition to enhancing the functional capacity [6]. TWT is said to upgrade the pavement from deteriorated condition to excellent condition and many pavements have been observed to stay in excellent/good condition even after 30 years of service [6]. TWT is also suggested for cases where good quality of pavement construction is guaranteed and traffic is restricted [7]. A number of methods are available for the thickness design of whitetopping and a few commonly used design methodologies are AASHTO Guide Method [8], Colorado method [9], PCA method [10], New Jersey method [11], Illinois method [12] and Texas method [13].

In India, IRC:SP:76-2015 [7] provides guidelines for the design of whitetoppings including TWT. Here, the joint spacings are limited between 1.0 and 1.5 m and hence, the primary distress considered is the corner breaking in PCC slabs. Separate equations are provided in IRC:SP:76-2015 to calculate the corner stresses due to load and curling. For calculation of curling, the temperature differential across the thickness of the slab is calculated depending on the location while the other parameters are related to the dimensions of the slab and its coefficient of thermal expansion. The traffic for design is specified in terms of the total commercial vehicles per day and the percentage of single and tandem axles in it. To account for the effect of load, individual equations are provided to compute the corner stresses for 8 tonnes single axle load and 16 tonnes tandem axle load. For other loads, the values are interpolated. A trial cross section is chosen and the corner tensile bending stresses are calculated as a function of the modulus of subgrade reaction, length of slab and radius of relative stiffness.

The fatigue damage progression in a concrete pavement is explained in terms of the stress ratio, which is the ratio of applied flexural stress to the flexural strength of concrete. If the stress ratio is less than 0.45, then concrete is said to sustain infinite number of repetitions and as the stress ratio increases, the number of load repetitions required to cause cracking decreases [14]. These fatigue criteria are used to estimate the allowable number of repetitions based on Miner's hypothesis. For a chosen thickness, the fatigue life consumed at the end of the design period is calculated and it should be less than 1. Otherwise, the thickness of the concrete overlay is varied accordingly.

The important aspect that needs to be addressed in the current procedure detailed in IRC:SP:76-2015 [7] is the effect of traffic. IRC:SP:76-2015 [7] suggests usage of TWT for medium to moderately heavy volume roads such as low-traffic National Highways (NH) and intersections among others. However, the traffic conditions for the same are not clearly defined. Traffic on a pavement is generally defined in terms of the magnitude of the axle load and number of repetitions of a given axle load. They have different effects on the TWT overlay. The magnitude of axle load is a critical factor as a single repetition of an overloaded axle can cause more damage to the pavement when compared with multiple repetitions of a standard axle load.

Axle overloading is a major concern in the design of pavements across the world. In the case of developed countries the extent of overloading is about 2–5%, while that in developing countries reaches about 80%. In India, considerable overloading of axles has been reported over a long time period [15–17]. Considering the magnitude of overloading observed in India, IRC:58-2015 suggests collection of axle load spectrum to estimate the number of repetitions of single, tandem and tridem axles for each load category at specified load intervals. The traffic considerations in IRC:58-2015 [14] are suggested for use in IRC:SP:76-2015. While many studies are available that estimate the effect of such axle load spectrum on the life of flexible pavements [18–21], such information is scarcely documented for TWTs.

IRC:SP:76-2015 specifies the use of a TWT for medium to moderately heavy volume roads such as Major District Roads (MDR), State Highways (SH) and low-traffic NH. However the suitable traffic conditions either in terms of the magnitude of the axle load or the number of repetitions for a given axle load are not specified. In this regard, the user is left with considerable ambiguity related to the traffic composition, especially the magnitude of axle loads, for which TWTs can be applied. It is therefore necessary to evaluate the performance of TWTs for the traffic conditions prevalent in India and specify the traffic characteristics in terms of the axle load spectrum for which TWTs can be used in India.

It should be noted here that the performance of TWT overlays, especially in case of heavier traffic loads, can be enhanced by the addition of fibres. Studies have shown that locations with limitation in thickness, those experiencing heavier traffic and those with increased joint spacing require the use of fibres in concrete mix [6]. Fibres have been observed to reduce the tendency to plastic shrinkage, increase ductility and abrasion resistance [22]. It has also been shown that use of structural steel fibres has increased the flexural performance and the ultimate load carrying capacity of concrete beams and slabs [23]. A number of options are available regarding the fibres. Though steel fibres have been in use for a while, synthetic fibres are preferred owing to their low density [24, 25]. The synthetic fibres are commonly made of polypropylene or polyester or nylon and are classified as micro- or macro-fibres depending on their diameter. The dosage of such fibres has to be optimised as the use of higher dosage of fibres is said to cause entanglement and result in a balling effect. This leads to a reduction in strength gain, especially for fibres with high aspect ratio [26].

Fibres, when used in concrete mixes, require addition of mineral admixtures in order to facilitate good bond between fibres and concrete [27]. While the mineral admixtures such as flyash, ground granulated slag and silica fume are recommended for this purpose [28], silica fume is commonly used. Silica fume, because of high reactivity with calcium hydroxide in concrete, results in a matrix that is very dense and has low permeability. This results in a pavement quality concrete that is cohesive, has high abrasion resistance and longer service life [29]. Higher dosage of silica fume, when added to concrete, makes the concrete brittle and hence ductility is a major concern [30, 31]. The area under the load–deflection curve became smaller with increase in silica fume content and it was seen to reduce its flexural toughness [30]. Hence the dosage of fibres and silica fume has to be optimised for each fibre type to achieve a mix with maximum load carrying capacity.

In this study, the traffic data were collected from four NH in the country and the TWT design was carried out based on the procedure specified in IRC:SP:76-2015. The effect of traffic on the fatigue life of TWTs prepared with conventional concrete and fibre-reinforced concrete was considered. Different dosages of fibres and silica fume were considered to determine the optimum combination of these factors for maximum load carrying capacity. It was seen that the mix with 1.8% fibres and 8% silica fume resulted in the maximum equivalent flexural strength. This mix and the mix without fibres and silica fume were considered to evaluate the effect of traffic. Three different limiting traffic conditions were considered here wherein, in the first case, the actual axle load was used while in the second, the modal axle load was used and in the third, the axle load closer to the legal axle load was used. It was seen that both the mixes were not safe for the actual traffic load observed for NH-58 (highway with heavier axles) while for NH-79 (highway with lesser amount of overloading), the mix with fibres resulted in a safe design. When the axle loads closer to those specified in IRC:3-1983 [32] were considered for design, it was seen that even the mix without fibres could sustain the design traffic for both the highways.

2. Experimental investigation

2.1 *Materials*

The cement concrete mix design for pavements adopted in this study was carried out based on the flexural strength requirement specified in IRC:44-2017 [33]. Based on IRC:SP:76-2015, a minimum flexural strength of 4.5 N/mm^2 is required for TWT and hence the same is used in this study. For the control mix, the water/cement (w/c) ratio adopted was 0.32 to achieve a workability corresponding to 25 mm slump. To compensate for the lower w/c ratio, IRC:SP:76-2015 suggests addition of admixtures of dosage up to 2% by mass of cementitious material. In this study, the admixture chosen was a super-plasticiser and a dosage of 1% by mass of cementitious material was added. A 20% reduction on the total water content was carried out to compensate for the addition of super-plasticiser.

To the cement concrete mix, wave type macro-polypropylene fibres as shown in figure 1(a) were added. The fibre quantity was determined based on the specifications provided in IRC:SP:46-2013. The melting point of these fibres was in the range 160–170 °C. In addition, silica fume of two different dosages is also added. When silica fume is added the w/c ratio is expressed as the water–cementitious material ratio (w/c-m). The silica fume used in this study is shown in figure 1(b). The description of the other materials used for preparation of the mix is given in table 1.

Six different mixes were prepared using combinations of the fibre dosage (0.8%, 1.3% and 1.8%) and silica fume content (8% and 13%). IRC:SP:76-2015 specifies addition of fibres in the range of 0.5–1.5% by the volume of concrete and silica fume in the range of 3–10% by weight of cementitious material. Two dosages were chosen within this range and one higher dosage was chosen to ascertain if any balling effect led to reduction in strength gain at such dosages. Similarly, for silica fume, one dosage (8%) was chosen in the range specified by IRC:SP:76-2015 and one higher dosage (13%) was chosen. The 13% dosage was chosen to evaluate if excessive silica fume content in fibre-reinforced concrete led to any reduction in strength gain or it aided better reinforcement of fibres at higher dosages, especially 1.8%.

In addition, a control sample with no fibres and silica fume was also prepared. To quantify the influence of silica fume independently on the strength of the mix, one mix was prepared with 8% silica fume and no fibers. Based on the results from this mix, addition of 13% silica fume and no fibres was not considered. Table 2 shows the composition of the eight mixes. The quantities given here are computed for 1 m^3 of concrete. Here, the fibre dosage is expressed in percentage by volume of concrete and silica fume is expressed in percentage by weight of cementitious material. Six specimens were cast for each mix shown in table 2.

2.2 *Testing of specimen*

Residual flexural strength and toughness tests were performed on the prisms as per IRC:SP:46-2013. The loading rate was maintained such that the peak load was achieved within 3–8 minutes from the start of loading. Once the load begins to drop after the peak, the loading rate is increased so as to finish the test in about 30 minutes. The test was conducted until the specimen reached a deflection value of span length/150, i.e. 2.4 mm for a span length of 360 mm.

To measure the maximum deflection in flexural strength test, a fixture with adjustable height was fabricated as shown in figure 2(a). A dial gauge is attached to the frame, which is fixed at the neutral axis of the prism. Another handle is fixed to the prism at the midpoint, which exactly rests on the surface of the prism. The dial gauge now measures the maximum deflection of the top surface of the

(a) Macro-polypropylene fibre. **(b)** Silica fume.

Figure 1. Photographs of materials used for preparation of concrete mix.

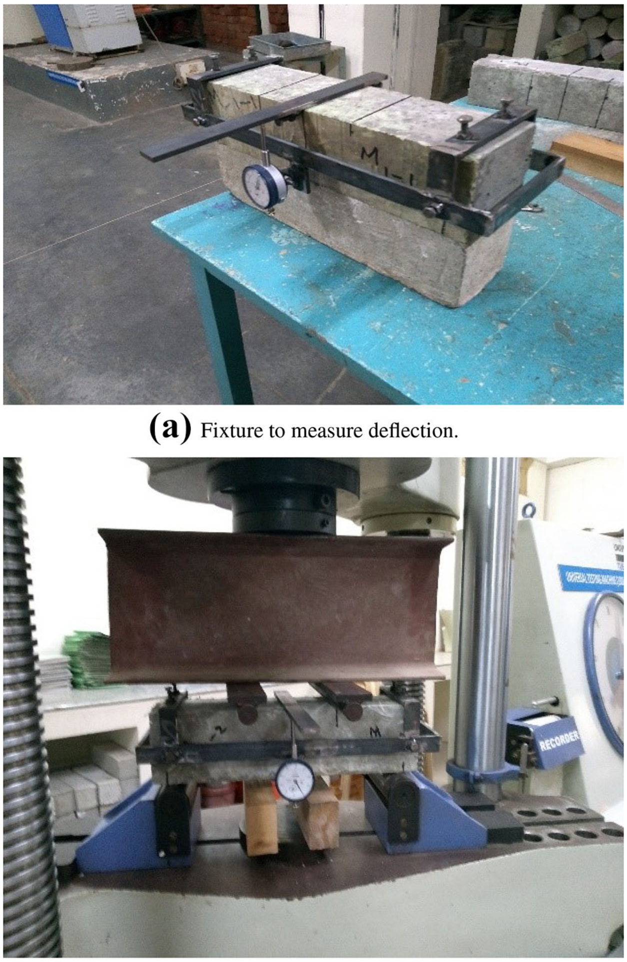
Table 1. Materials specifications.

Sl. no.	Material	Description
1	Fibre	Macro-polypropylene fibres with aspect ratio 40–60, length 50 mm, density 910 kg/m^3
2	Silica fume	Micro-silica conforming IRC:114-2013
3	Cement	Portland slag cement conforming IS:455
4	Water	Potable water
5	Superplasticiser	Fosroc Conplast SP430
6	Fine aggregate	Conforming grading zone II as per IRC:44-2017, Specific gravity -2.63
7	Coarse aggregate	Maximum nominal size of 20 mm, Specific gravity -2.71

Table 2. Mix composition for flexural strength test.

Mix no.	Volume of fibres in terms of total concrete volume (%)	Mass of silica fume in terms of mass of cementitious materials (%)	Mass of fibres (kg)	Mass of silica fume (kg)	w/c / w/c-m ratio	Mass of cement (kg)	Mass of water (kg)	Mass of fine aggregate (kg)	Mass of coarse aggregate (kg)	Mass of superplasticiser (kg)
M1	0	0	0	0	0.32	450	144	633	1243	4.5
M2	0.8	8	7.28	39.17	0.30	450	144	605.54	1222	4.89
M3	1.3	8	11.83	39.17	0.30	450	144	605.54	1222	4.89
M4	1.8	8	16.38	39.17	0.30	450	144	605.54	1222	4.89
M5	0.8	13	7.28	63.65	0.30	426	144	602.7	1216.4	4.89
M6	1.3	13	11.83	63.65	0.30	426	144	602.7	1216.4	4.89
M7	1.8	13	16.38	63.65	0.30	426	144	602.7	1216.4	4.89
M8	0	8	0	39.17	0.30	450	144	605.54	1222	4.89

sample at mid span. The test set-up is shown in figure 2(b). Failure by pure bending was considered only when the specimen failed by cracking in the middle one-third portion. Cracks in any other region are said to be a result of shear failure and such samples were omitted for further analysis.



(a) Fixture to measure deflection.

(b) Flexural strength test set-up.

Figure 2. Flexural strength test on cement concrete prisms.

2.3 *Traffic data*

The traffic count and axle load data collected by M/s V R Techniche for four NH were used in this study. The traffic data were collected for 7 days and 10% of the trucks were sampled for axle load distribution data. The locations at which the data were collected and the corresponding annual average daily traffic (AADT) observed at these locations are shown in table 3.

3. Results and discussion

3.1 *Residual flexural strength*

Figure 3 shows the load vs. deflection curve as an instance for mixes M1, M8, M2 and M4. For the mixes without fibres, M1 and M8, it is seen that the failure occurs at peak loads of 15.8 and 7 kN, respectively (figure 3(a)). The failure is observed to be brittle for both the cases due to the absence of fibres. For the mix with fibres, two cases M2 and M4 with identical silica fume content of 8% are compared as shown in figure 3(b); for these mixes, the peak loads are observed at 10.1 and 10.2 kN, respectively. After the peak load, a sudden drop in load is observed as shown in figure 3(b) and the load begins to increase further. The increase in load is more prominent in the case of mix M4, which has higher fibre content compared with mix M2. Such increase in load is attributed to the strain hardening effect and it is said to be more prominent in the case of higher fibre dosage [34–36].

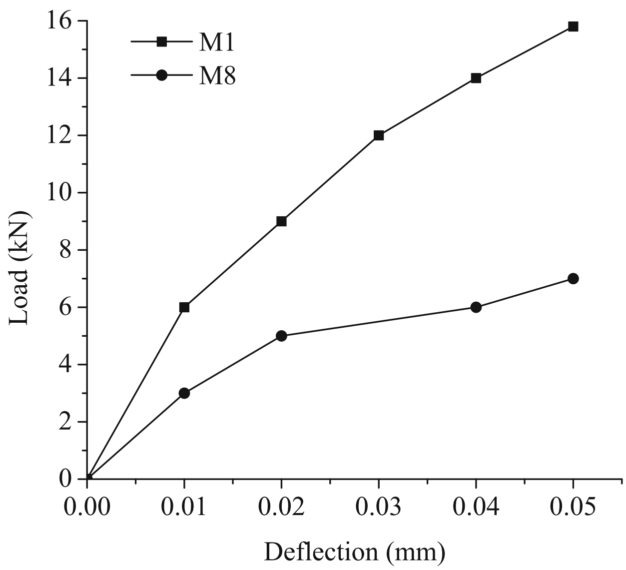
Two parameters are observed from the load vs. deflection curve. The first is the peak load (P_{max}, N), which is the first point on the load–deflection curve where the slope is zero. The peak flexural strength (f_{ct}, N/mm^2) is calculated as ($P_{max} \times l$)/bd^2, where l, b and d are length, breadth and height of the specimen, respectively, in mm. The second is the equivalent flexural load (P_{e150}, N), which is the average capacity in the post-peak region up to a specified deflection of l/150. It is calculated by dividing the total area under the load–deflection curve by the specified deflection (l/150), which is 2.4 mm for the span 360 mm used here. The equivalent flexural strength (f_{e150}, N/mm^2) is calculated as ($P_{e150} \times l$)/bd^2.

The total flexural strength (f_{total}, N/mm^2), in the case of fibre-reinforced concrete, is the summation of peak flexural strength (f_{ct}, N/mm^2) and equivalent flexural strength (f_{e150}, N/mm^2). The total flexural strength is not defined as per IRC:SP:76-2015 or as per IRC:SP:46-2013. However, this parameter is calculated here by a slight rearrangement of the terms; the stress ratio defined in IRC:SP:76-2015 is seen to be the ratio of the flexural stress to the sum of peak and equivalent flexural strength, which is indicated here as total flexural strength. Residual strength ratio (R_{150}) is calculated as the ratio of equivalent flexural strength (f_{e150}, N/mm^2) to peak flexural strength (f_{ct}, N/mm^2). Table 4 shows the average peak flexural strength, equivalent flexural strength, total flexural strength and residual strength ratio for the 8 mixes. The values given in table 4 are the mean values for the 6 specimens cast for each mix. It is seen that M4 exhibits the highest total flexural strength and M1 exhibits the least total flexural strength.

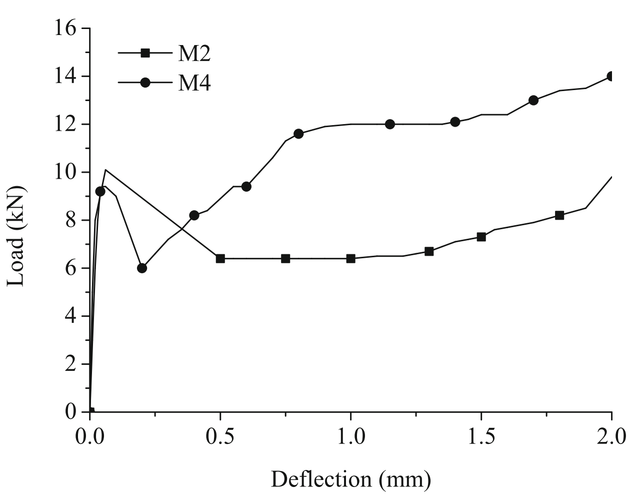
On comparing M1 and M8 it is seen that by adding 8% silica fume to control mix (M1), there is a 51.9% reduction in flexural strength from 4.93 to 2.37 N/mm^2. Addition of only silica fume to the mix makes it brittle, leading to a reduction in strength. A similar behaviour is also reported in the literature [30, 31]. Hence addition of 13% silica fume without fibres was not considered. Table 4 shows that, for 8% silica fume (M2, M3, M4), when fibres are added, the total flexural strength increases with increase in fibre content. Both f_{ct} and f_{e150} are seen to increase with fibre dosage for 8% silica fume when compared with M8. As the fibre dosage increases to 1.8%, it is also seen that f_{e150} is marginally higher than f_{ct} due to the strain hardening effect indicated in figure 3(b). However, for addition of 13%

Table 3. Annual average daily traffic for different highways.

Highway	Data collection point	AADT (CV/day)
NH-13	318 km in the Hospet–Chitradurga stretch	6975
NH-15	South bound direction at 133th km	2615
NH-58	Shivaya toll plaza	6639
NH-79	189th km in the South bound direction	6230



(a) Mix without fibres.



(b) Mix with fibres.

Figure 3. Load–deflection curves for selected mixes.

silica fume (M5, M6, M7), the total flexural strength reaches a maximum at 1.3% fibre content and reduces for 1.8% fibre content. For M7, it can be seen that both f_{ct} and f_{e150} are lower as compared with M6.

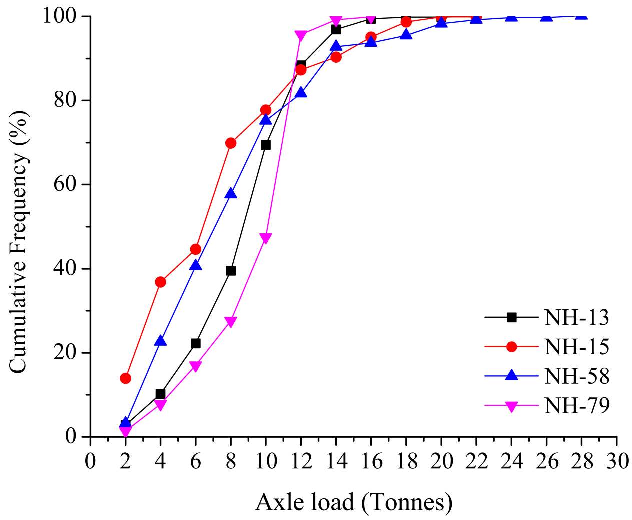
Table 4. Total flexural strength results for different mixes.

Mix	f_{ct} (N/mm^2)	f_{e150} (N/mm^2)	f_{total} (N/mm^2)	R_{150}
M1	4.93	0	4.93	0
M2	2.97	2.27	5.24	0.76
M3	3.05	2.27	5.32	0.74
M4	3.38	3.44	6.82	1.02
M5	2.96	2.21	5.17	0.75
M6	3.16	3.22	6.38	1.02
M7	2.8	3.1	5.90	1.11
M8	2.37	0	2.37	0

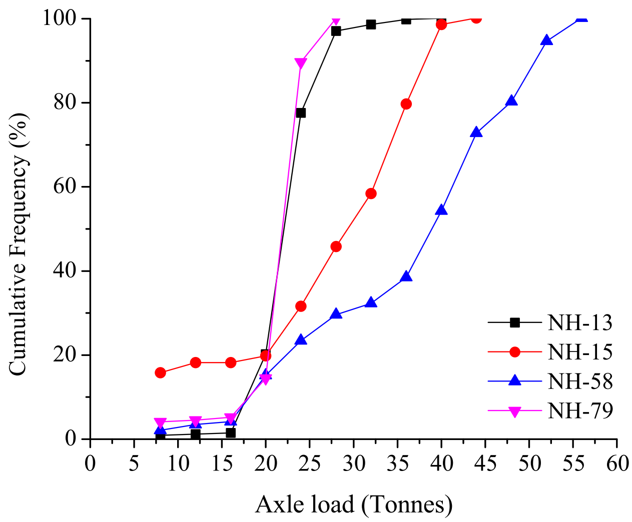
To compare the influence of silica fume, mixes with identical fibre content M2 and M5 can be compared. It is seen that with increase in silica fume content, a reduction in f_{e150} is observed for 0.8% and 1.8% fibre content. However, for 1.3% fibres (M3 and M6), it is seen that an increase in f_{e150} is observed with increase in silica content. It can be said that when silica is present in an optimum range, it contributes effectively to improvise the bond between fibres and concrete. For lower silica content, the silica can be said to be insufficient to create bonding while excess silica present can lead to reduction in strength gain. Hence, for maximum strength gain with fibres, the quantity of silica fume has to be optimised. The control mix M1 and mix M4 with maximum total flexural strength were considered to evaluate the effect of traffic.

3.2 *Analysis of traffic data*

Figure 4 shows the axle load distribution vs. frequency for different highways. If the single axle loads of different highways are compared (figure 4(a)), it can be seen that the axle load ranges are 2–18 tonnes for NH-13, 2–20 tonnes for NH-15, 2–28 tonnes for NH-58 and 2–16 tonnes for NH-79. Among these 4 highways, if NH-13 and NH-15 are compared, it is seen that the axle load range remains almost identical for both the highways while the distributions are different. For NH-15, it is seen that the cumulative frequency is higher when compared with NH-13 up to 12 tonnes and then the trend is reversed. This shows that the proportion of axle loads greater than 20 tonnes is higher in



(a) Single axle.



(b) Tandem axle.

Figure 4. Comparison of axle load vs. frequency for different highways.

NH-13 when compared with NH-15. If the single axle load distribution for NH-13 and NH-58 is compared, it is seen that the axle load distribution remains close for both the highways up to 22 tonnes. However, beyond 22 tonnes, 5% of axles are seen for NH-58 while no axles are seen for NH-13. The presence of heavier axle loads for NH-58 compared with all other highways can be seen from figure 4(a) for the single axles and figure 4(b) for the tandem axles.

For the tandem axles (figure 4(b)), the axle load ranges are 8–40, 4–44, 8–56 and 8–28 tonnes for NH-13, NH-15, NH-58 and NH-79, respectively. It is seen that similar to the single axle load distribution, tandem axle load distribution for NH-13 exhibits higher percentages of heavier axle load groups when compared with NH-15. NH-79 consists of 100% axle loads less than 16 tonnes for the single axle load and less than 28 tonnes for the tandem axle load. Comparison of the design for NH-13 and NH-15 will exhibit the influence of frequency distribution for identical axle load ranges while comparison of design for NH-13 and NH-58 will exhibit the influence of axle load distribution skewed towards right. The effect of different traffic characteristics on the design of TWT is discussed in the next section.

3.3 *Effect of traffic on design of TWT*

The effect of traffic on the design of TWT was analysed based on design guidelines provided in IRC:SP:76-2015. Here, the analysis was carried out individually to assess the impact of different axle load distributions and the number of repetitions of selected axle load groups. The design life is considered as 20 years for this purpose. A pavement thickness of 18 cm is considered with panel size 120 cm $\times$ 120 cm. The modified modulus of subgrade reaction is considered to be 10 kg/cm^3; the flexural strength of concrete is 4.93 N/mm^2 for M1 and 6.82 N/mm^2 for M4 as per table 4. The effects of axle load distribution and AADT are discussed individually in the following.

3.3.1 *Impact of axle load distribution* To compare the influence of axle load distribution, an AADT of 4000 was considered for the three highways NH-13, NH-15 and NH-58. It should be noted that the AADT value used here differs from the actual AADT shown in table 3. A uniform value of 4000 was used to compare the effect of axle load distribution on the fatigue damage. The fatigue damage for the single and tandem axles are shown in table 5. From this table 5, it is observed that the single axle fatigue for NH-13 is 0.9 while the same for NH-15 is 2.9. The single axle fatigue in NH-15 is higher by 222% when compared with NH-13. Similarly, the tandem axle fatigue for NH-15 is 9613% higher when compared with NH-13. It can thus be said that highways with higher frequency of heavier axle loads contribute exponentially to the fatigue damage. Based on the comparison of single axle traffic loads between NH-13 and NH-58 shown in table 5, it is observed that the fatigue damage for NH-58 is higher by 113591%. This shows that the magnitude of load plays a major role in determining the fatigue damage.

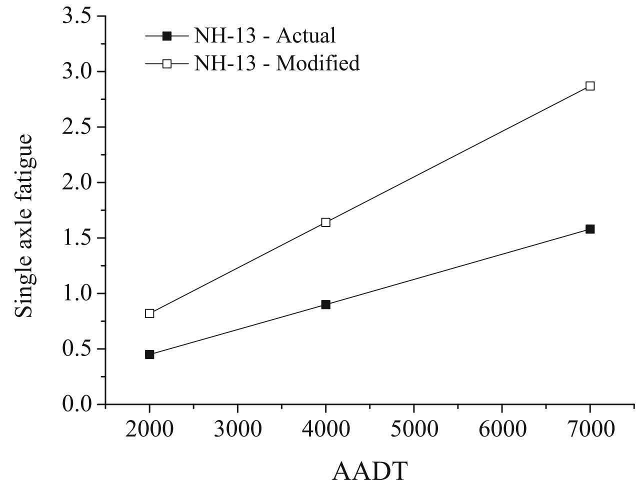
Table 5. Fatigue damage for NH-13, NH-15 and NH-58.

Parameter AADT	National Highway		
	NH-13	NH-15	NH-58
Single axle fatigue	0.9	2.9	1023.22
Tandem axle fatigue	81.9	7955.55	4235389.8

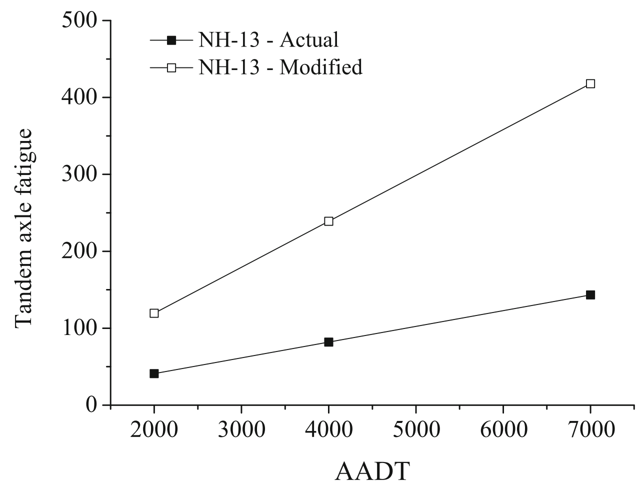
In order to analyse the sensitivity of magnitude of load, the axle load distribution of NH-13 is modified as shown in table 6. Here, one 20 tonne axle load was added to the single axle load distribution and one 40 tonne axle load was added to tandem axle load distribution of NH-13. This is shown as the modified axle load distribution for NH-13 (NH-13 M) in table 6 along with unmodified data (NH-13). It can be seen that the axle load count remains identical for all load groups except for the 20 tonnes axle load added to single axle load distribution and 40 tonnes axle load added to tandem axle load distribution. For these modified data, the single and tandem axle fatigues are recalculated and the results are shown in figure 5. For an AADT of 4000, it is seen that there is approximately 80% increase in single axle fatigue damage and 190% increase in tandem axle fatigue damage for addition of one overloaded axle. This shows that the TWT design considered here is highly sensitive to the magnitude of axle load.

3.3.2 *Impact of AADT* Fatigue damage in bituminous pavements is generally said to occur due to continuous repetition of load. While the effect of one repetition of overloaded axle on the fatigue damage has been established, it is now desired to estimate the magnitude of influence exerted by the variation in AADT. To quantify the effect of AADT, three different AADTs, namely 2000, 4000 and 7000, were considered for the three highways, NH-13, NH-15 and NH-58. These values are also hypothetically chosen and do not correspond to the values shown in table 3. Figure 5 shows the fatigue damage for different AADT values for NH-13. To see if the fatigue damage increase is proportional to the AADT increase, the cumulative fatigue damage is calculated.

The cumulative fatigue damage is calculated as the sum of the fatigue damage of single and tandem axles. The cumulative fatigue damage of all the three highways was normalised for an AADT of 2000. It was observed that for all the three highways, the effect of variation in AADT resulted in an identical variation in cumulative fatigue damage as shown in figure 6. Here, the normalised cumulative fatigue damage vs. AADT curves for all the three highways are seen to be super-imposed on each other. The proportion of the axle loads remains identical for any given AADT and since the design methodology focuses on the percentage of axles in each load group to estimate the fatigue damage, the normalised graphs for different AADTs are identical. It could thus be seen that the design procedure specified in IRC:SP:76-2015 exhibits higher sensitivity to the magnitude of axle load when compared with the number of repetitions of a standard axle load. In the following, the effect of axle load distribution for NH-58 and NH-79 based on the actual traffic data collected from field is discussed.



(a) Single axle - 20 tonne.



(b) Tandem axle - 40 tonne.

Figure 5. Effect of addition of one overloaded axle to NH-13.

Table 6. Modified axle load count for NH-13.

Axle load (tonnes)	Single axle (nos.)		Axle load (tonnes)	Tandem axle (nos.)	
	NH-13	NH-13 M		NH-13	NH-13 M
<2	8	8	<8	3	3
2–4	21	21	8–12	1	1
4–6	34	34	12–16	1	1
6–8	49	49	16–20	64	64
8–10	85	85	20–24	197	197
10–12	54	54	24–28	67	67
12–14	24	24	28–32	5	5
14–16	7	7	32–36	4	4
16–18	1	1	36–40	1	1
18–20		0	40–44		*1*
20–22		*1*			

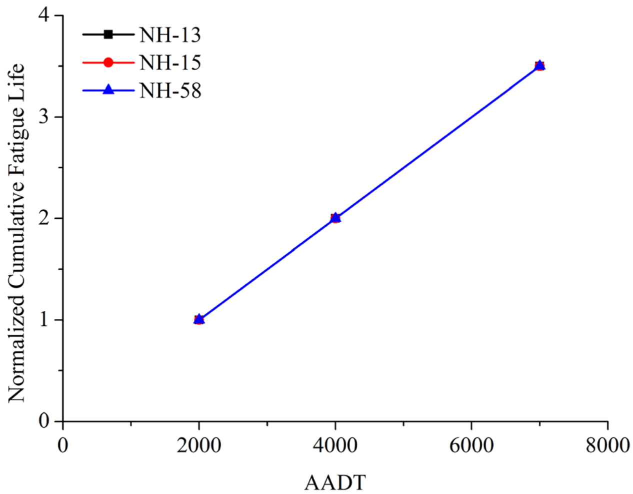


Figure 6. Normalised cumulative fatigue damage variation with AADT.

3.4 *Comparison of TWT design for NH-58 and NH-79*

To evaluate the effect of traffic conditions on the TWT design the traffic data collected from field for the two highways, namely NH-58 and NH-79 with identical AADT values, were taken into consideration. Table 3 shows that the AADT of NH-58 and NH-79 is 6639 and 6230, respectively. However, the axle load distributions for the two highways are completely different as shown in figure 4. It is seen that NH-58 carries heavier axle loads as compared with NH-79 irrespective of identical ranges of AADT. A uniform thickness of 180 mm was adopted for both the mixes, M1 and M4, for comparison purposes. It should be noted here that the thickness used here is 180 mm, which is closer to the limit of 200 mm specified for TWTs. The fatigue damage calculated for both the highways is shown in table 7.

Table 7 shows that when the mix M1 (without fibres) is used, a safe design is not obtained for the design variables used and the actual traffic conditions observed from field. The design is generally considered to be safe when the fatigue life consumed is less than one. For NH-79, when the AADT was reduced to 1735 for the same axle load distribution, it was seen that a safe design could be obtained. However, a safe design could not be obtained in the case of NH-58 even when the AADT was reduced up to one. Alternatively, when the mix with fibres (M4) is used, the design variables result in a safe design for the actual AADT observed in NH-79. For this mix, one could obtain a safe design when the AADT was reduced to 45 in the case of NH-58. Two different scenarios that could result in a safe design for the AADT and axle load distribution observed in field were then analysed.

As the first scenario, the axle loads for single and tandem axles were limited to 14 and 28 tonnes, respectively, for both NH-58 and NH-79. This is carried out by removing the axle loads greater than 14 and 28 tonnes from the axle loads collected from field and the axle load distribution is recalculated. These loads are chosen as they are observed to be the most commonly occurring axle load limits considering all the four highways as seen from figure 4. For NH-79 and NH-58, the design is carried out based on the recalculated axle load distribution and the design parameters are shown in table 7. For this case also, the mix without fibres was not safe for both the highways. However, by

Table 7. Comparison of NH-58 and NH-79 for different traffic conditions.

		NH-79			NH-58		
Concrete mix	Condition	AADT	Cumulative fatigue	Design	AADT	Cumulative fatigue	Design
				Actual traffic			
Without fibres (M1)	Actual	6230	3.59	Not safe	6639	7031387	Not safe
	Minimum	1735	0.99	Safe	1	1059.1	Not safe
With fibres (M4)	Actual	6230	0	Safe	6639	145.064	Not safe
	Minimum	–	–	–	45	0.98	Safe
				Mostly observed traffic (SA≤14&TA≤28)			
Without fibres (M1)	Actual	6230	3.58	Not safe	6639	4.88	Not safe
	Minimum	1741	0.99	Safe	1360	0.99	Safe
With fibres (M4)	Actual	6230	0	Safe	6639	0	Safe
	Minimum	–	–	–	–	–	–
				Standard traffic (SA≤8&TA≤16)			
Without fibres (M1)	Actual	6230	0	Safe	6639	0	Safe
	Minimum	–	–	–	–	–	–
With fibres (M4)	Actual	6230	0	Safe	6639	0	Safe
	Minimum	–	–	–	–	–	

reducing the AADT to 1741 and 1360, one could obtain a safe design for NH-79 and NH-58, respectively, with mix M1. When mix M4 is used, a safe design is obtained for the actual AADT observed in field for both the highways. In the second scenario the axle loads were limited to 8 tonnes for single axles and 16 tonnes for tandem axles, closer to the ranges specified by IRC:3-1983 [32] and other international specifications. For such a case, it was seen that the mix M1 was sufficient to obtain a safe design for both the highways for the actual traffic conditions discussed earlier.

4. Summary and conclusion

In this study, an attempt was made to estimate the suitability of TWT for Indian conditions based on the traffic data observed from four NH in India. For this purpose, the whitetopping mix was prepared with polypropylene fibres and silica fume was added to this mix for good bond between fibres and concrete. The design procedure for TWT specified in IRC:SP:76-2015 was adopted. The traffic data collected from four NH in India were compared using actual axle load data and hypothetically varying them for a few cases. It was seen that the fatigue damage varied exponentially with axle load whereas the variation in AADT resulted in a linear variation in fatigue response.

The specific conclusions based on the analysis carried out in this study are as follows:

1. The silica fume and the fibre dosage are not independent of each other. For higher silica fume content, the excess silica results in a reduction in strength gain; however, for lower silica content, good dispersion of fibres is not achieved. For an optimum silica fume content the fibres are dispersed in concrete, resulting in sufficient strength gain. In this study, the mix M4 with 1.8% fibres and 8% silica fume resulted in the maximum total flexural strength.
2. The fatigue damage of TWT overlay for bituminous pavements based on IRC:SP:76-2015 is highly sensitive to the magnitude of load. It was seen that for increase in one single axle load of 20 tonnes and one tandem axle load of 40 tonnes, the fatigue damage increased by 80% and 190%, respectively. This number is expected to increase exponentially with increase in magnitude of axle load.
3. When AADT was taken into consideration, the fatigue damage increase was seen to be proportional to the increase in AADT irrespective of the axle load distribution. Though the fatigue damage is generally related to the number of repetitions, it is seen that the TWT design procedure considered here is sensitive to the axle load distribution. The proportion of axles in each axle group is constant for a given highway and it increases proportionately with increase in AADT. This results in a linear variation of fatigue damage with increase in AADT.
4. For the actual traffic observed in NH-79 and NH-58, a safe design could not be obtained for the conventional mix. Even when the mix with fibres was used, a safe design could not be obtained for NH-58. The possibilities that could result in a safe design were analysed for both the highways with the two different mixes. When the single axle load was limited to 14 tonnes and tandem axle load to 28 tonnes, a safe design was observed for both the highways when the mix with fibres was used. When the single and tandem axle loads were limited to 8 and 16 tonnes, it was seen that the mix without fibers could be also used to sustain the traffic. Thus depending on the axle load distribution, a mix with fibres can be sufficient for certain cases while for certain cases, the axle loads have to be limited even when the use of fibres is considered.

It is thus seen that the design of TWT as per IRC:SP:76-2015 is highly sensitive to the magnitude of axle load rather than the number of repetitions. If the conventional mix without fibres is considered, it can be used only in locations where the single and tandem axle loads are limited to 8 and 16 tonnes or the AADT is in the range of 1300–1700 with single and tandem axle loads limited to 14 and 28 tonnes, respectively. If the AADT is higher (say about 6000), it should be ensured that the single and tandem axle loads observed in field should be again limited to 14 and 28 tonnes, respectively, and the use of fibres is considered.

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