

Influence of geosynthetic-interlayers on the performance of asphalt overlays on pre-cracked pavements

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Abstract

The functions of geosynthetic-interlayers in retarding reflection cracking and improving fatigue performance of hot mix asphalt (HMA) overlays in flexible pavements are evaluated in this study. The delamination or debonding mechanisms of the overlays are studied when geosynthetic-interlayers are adopted. A polyester grid coated with polymer modified binder (G1), a woven geo-jute mat (G2), and a bi-axial polypropylene grid (G3) interlayer are examined based on their adhesion properties. A two stage experimental program is reported. During the first stage, the performance of the geosynthetic-interlayers sandwiched between the pre-cracked old pavement and new asphalt layers are evaluated using flexural fatigue testing. A digital image correlation (DIC) technic was employed to record the failure modes and the corresponding tensile strains in the overlay system. During the second stage, the effect of interlayers on the interface bond strength was evaluated with the help of shear and tensile bond strength tests. The results show that the inclusion of interlayers retard the propagation of reflection cracking, however, results in the delamination of overlays. The debonding effect is prominent in G3 interlayers due to their high initial stiffness. Overall, interlayers with high interfacial shear and pull-off tensile bond properties proved effective in controlling the reflection cracking and increasing fatigue life of the overlays.

Keywords: *Geosynthetics, HMA overlays, reflection cracking, flexural fatigue testing, interface bond strength.*

1. INTRODUCTION

The hot mix asphalt (HMA) overlays, generally referred to as asphalt overlays, is the common and convenient rehabilitation technic available to restore the serviceability of the existing pavement surface. It is often observed that these overlays are associated with reflection cracking problems. Reflection cracking may be defined as a process of propagation of discontinuities or cracks from the existing distressed layer through the new overlay under traffic and temperature induced stresses (Cleveland et al. 2002; Kim and Butlar, 2002). The thickness of the overlay is also one of the factors responsible for reflection cracking (Goulias and Ishai, 1999). Reflection cracking causes premature failure of overlays, and then the entire pavement system by allowing moisture into the subsequent layers underneath via cracks (Elseifi and Al-Qadi, 2003; Farshad, 2005; Smith, 1983). The reflection cracking is a very complex phenomenon, which cannot be completely arrested, however, various technics are available to resist the crack propagation. The most common technic is to provide an interlayer at the interface of an old and new pavement layers. In general, fabrics or geotextiles, geogrids, composites and stress absorbing membrane interlayers (SAMI) are adopted for this purpose. The interlayers improve the performance of the overlays by providing stress relief, reinforcement and moisture control (Elseifi, 2003). However, the interlayer should possess sufficient cross-sectional area and modulus to improve the strength and performance of the overlays (Lytton, 1989). Hence, a proper understanding of the behavior of interlayers in retarding the reflection cracks is necessary to select an appropriate interlayer for a given pavement system.

22

1 **2. BACKGROUND**

2 A large number of laboratory, field and numerical studies were performed to evaluate the
3 effectiveness of various interlayer systems to retard the reflection cracks. The location of an
4 interlayer in a pavement system is crucial in absorbing tensile strains mobilized during
5 loading. When placed at the interface of an old and new overlay, interlayers effectively
6 absorb the tensile strains and reduces the vertical deformation and there by the crack
7 propagation (Brown et al. 2001; Khodaii et al. 2009; Li et al. 2016; Moayedi et al. 2009).
8 The fatigue life, bending strength and crack-resisting potential was observed to improve
9 when the fiberglass grid interlayer was placed at the bottom of asphalt layer (Guo and Zhang,
10 1993; Fallah and Khodaii, 2015; Virgili et al. 2009). A paving fabric or geotextile interlayer
11 system, which provides stress relief and moisture control functions, was found effective in
12 improving the pavement performance (Farshad, 2005). The fabric interlayers were also found
13 cost-effective when compared to the thick asphalt overlays (Steen, 2004). The inclusion of
14 geosynthetic-interlayers not only improves the performance of HMA overlay in terms of
15 reducing the rate of reflective cracking in overlays but also increase the resistance to rutting
16 (Correia and Zornberg, 2016; Goulias and Ishai, 1999; Khodaii et al. 2009). The effects of
17 geosynthetic-interlayers on reflective crack retardation were evaluated under laboratory
18 conditions by Caltabiano (1990), El Meski and Chehab (2013), Gonzalez-Torre et al. (2015),
19 Sanders et al. (1999) and Walubita et al. (2015). They found that the interlayers improved
20 the performance life of the overlays effectively by resisting the crack propagation. Ogundipe
21 et al. (2012) found SAMIs to be effective in retarding the crack propagation only at low load
22 levels. Similarly, Kim et al. (2010) and Pasquini et al. (2014, 2015) found that the appropriate

1 selection and application of optimized grid-reinforced bituminous membrane (composites)
2 can significantly enhance the reflective cracking resistance in asphalt pavements. Barraza et
3 al. (2010) and Nejad et al. (2016) evaluated the durability of various anti-reflective cracking
4 systems including geotextiles, geogrids and SAMI layers under cyclic loads and found that
5 all the interlayer types reduced the reflective cracking process and geogrids were found to be
6 most effective. A similar type of result for grids was obtained by Brown et al. (2001) and
7 Canestrari et al. (2013); however, Brown et al. (2001) and Pasquini et al. (2014, 2015)
8 observed that there was a reduction in shear resistance for specimens with geosynthetic-
9 interlayers. The reduction in the shear resistance was observed to be minimal in the case of
10 geogrids and the reason was attributed to the presence of apertures. On the other hand,
11 Barraza et al. 2010; Brown et al. 2001; and Caltabiano and Brunton, 1991 have shown that
12 the interlayers without the apertures (geotextiles) have proven least shearing resistant. The
13 interlayer bonding is found to be improved when interlayers are coated with an appropriate
14 binder or activated epoxies (Aldea and Darling, 2004; Ferrotti et al. 2012).

15 From the literature, it can be often noted that the geosynthetic-interlayers are placed in a
16 two layer (new asphalt layers), i.e. the interlayers were provided between the two new asphalt
17 layers, but not at the interface of an old and new asphalt layers. However, the general practice
18 is to place the interlayer with an appropriate tack coat at the interface. In addition, a mixed
19 performance of the interlayers was witnessed, when placed in a new asphalt overlay. Hence,
20 the current study aims to replicate the field scenario in the laboratory while testing an aged
21 and cracked old pavement layer, which was extracted from a distressed pavement, reinforced
22 with geosynthetic-interlayers and asphalt overlays. The objective is to evaluate the

1 performance of the interlayers against the crack propagation into the overlays under cyclic
2 four point bending tests with the aid of digital image correlation technic. The interface
3 bonding of interlayers with the old and new pavement layers is also evaluated to understand
4 the various parameters affecting the interface bond strength.

5 **3. MATERIALS AND SAMPLE PREPARATION**

6 To prepare two-layer asphalt beams with and without interlayers shown in Fig. 1, asphalt,
7 tack coat (binder) and different types of geosynthetic-interlayers, shown in Fig. 2, were used.
8 Following sections discuss the index and engineering properties of these materials.

9 **3.1. Asphalt**

10 The asphalt, also known as asphalt concrete and bituminous concrete, consists of a
11 bitumen binder and aggregates and a filler material which are mixed thoroughly in a mixing
12 plant. Two-layer asphalt concrete beams consists of a new asphalt concrete overlay
13 compacted over an old pavement layer as shown in Fig. 1. The old pavement's surface layer
14 was cut and carefully extracted from an existing state highway during the rehabilitation
15 process. The old asphalt concrete cake was cut into required sizes for the testing. The upper
16 layer consisted of asphalt concrete with a nominal aggregate size of 13 mm. The asphalt
17 concrete mix was prepared in the mixing plant and transported to the laboratory. The mix
18 comprises of a penetration grade (PG) 60/70 bitumen with an optimum binder content of
19 5.5% by weight of the aggregate as determined by Marshall-stability test. The HMA concrete
20 has a maximum strength of 14.25 kN and a flow value of 2.5 mm as per the Marshall-stability
21 test.

1 **3.2. Binder tack coat**

2 Tack coat materials generally employed are either bitumen emulsions or cutbacks,
3 whereas, the bituminous emulsions are the most commonly used tack coat material in both
4 rehabilitation and new projects. The application of asphalt cement as a tack coat at the
5 interface with geosynthetic-interlayers was proved to improve the interface bond strength
6 (Button and Lytton 1987). Hence, the asphalt binder of the same grade as that used in the
7 HMA overlay was used as a tack coat in this study. The physical properties of the tack coat
8 are presented in Table 1. The viscosity of the binder was determined in a Brookfield
9 viscometer at 60 °C. Based on the penetration value of 66, the binder tack coat is classified
10 as a bituminous binder of PG 60/70.

11 **3.3. Geosynthetic-interlayers**

12 Three types of geosynthetic-interlayers viz. a polyester grid, a woven jute mat and a
13 biaxial polypropylene grid were considered in the current study (Fig. 2). These materials
14 were selected based on the material type, aperture size and other tensile characteristics to
15 investigate their influence in interface shear, tensile bond strength and fatigue characteristics.
16 A wide width tensile strength test was performed according to ASTM D4595 (2011) on the
17 geosynthetic-interlayers on both machine direction (MD) – which would be along the length
18 of the road, and cross- machine direction (CMD) - across the width of the road. These tests
19 were conducted initially at room temperature (25 °C) and repeated after a temperature test.
20 In the temperature tests, the geosynthetic-interlayers were heated up to 150 °C for 2 hrs. and
21 then reduced to room temperature for 24 hrs. The latter exercise is to verify the influence of
22 temperature on the working properties of the geosynthetics which are typical during the

1 construction process, where the typical temperature of the asphalt concrete would be in the
2 range of 135 °C – 150 °C. Figures 3a and 3b show the variation of the tensile strength of the
3 geosynthetic materials obtained at 25 °C and after the temperature test respectively along
4 machine and cross machine directions. The properties of geosynthetic-interlayers used in the
5 study are as follows:

6 *Polyester grid (G1)*: The grid is manufactured using a high molecular weight and high
7 tenacity polyester yarns. The high tenacity yarns are knitted to form a geogrid material having
8 an aperture size of 18 mm as shown in Fig. 2a. The polyester grid is coated with a polymeric
9 modified bitumen layer. It is expected that the polymeric modified bitumen layer offers a
10 high bondage between the old and new asphalt layers. The polyester grid is a 2 mm thick
11 material (including the bitumen coating) with a square aperture opening of 18 mm. The
12 tensile strength of the polyester grid (G1) is about 48 kN/m and 52 kN/m along MD and
13 CMD respectively, however, there is no considerable reduction in tensile strength after the
14 heating test. The failure strain of this material is observed to be about 18-20% at room
15 temperature, and it has increased to 32-34 % after the temperature test (see Fig. 3).

16 *Woven geojute mat (G2)*: The mat is manufactured out of natural jute materials like fibers
17 and/or threads. The fibers are woven naturally or by the machine to form a mat without any
18 apertures as shown in Fig. 2b. The tensile strength of the material is 25 kN/m (MD) at 5%
19 strain and 20 kN/m (CMD) at 13% strain (Fig. 3). The influence of temperature is found to
20 be negligible on the geojute mat.

21 *Biaxial polypropylene grid (G3)*: The biaxial grid is made up of 4 mm thick polypropylene
22 material with a 40 mm square aperture (Fig. 2c). The tensile strength of the biaxial grid is

1 about 42 kN/m (CMD) and 36 kN/m (MD) at 10-12% failure strain. The G3 interlayers have
2 shown a drastic reduction in ultimate tensile strength of about 29% in machine direction and
3 37% in cross machine direction, without any change in the initial stiffness.

4 **3.4. Two-layer asphalt concrete specimen preparation:**

5 The two-layer asphalt concrete specimens for repeated load four-point bending tests were
6 prepared using a static weight compactor weighing 5 kg having a constant height of fall of
7 50 cm. The sample preparation consisted of placing an old pavement layer of size 400 mm
8 (length) \times 300 mm (width) \times 45 mm (thickness) as a bottom layer in a steel mold. Then a
9 bitumen of PG 60/70 was applied at a residual rate of 0.25 kg/m² on the old pavement layer
10 as a tack coat and allowed for emulsion breaking as per the ministry of road transport and
11 highways (MORTH) specifications (MORTH, 2003). Then, in the case of controlled
12 specimens, which are referred as NG specimens, a 45 mm thick HMA layer was compacted
13 over the old pavement block. The air void content in the HMA layer was theoretically
14 calculated as 7%, which is slightly above the general range of 3-6% for asphalt concrete
15 layers. The difference may be attributed to the compaction methodology adopted in this study
16 against the gyratory compaction. In the case of geosynthetic-interlayered specimens, the
17 interlayer was placed after the emulsion breaking time, which was obtained as 30 min in prior
18 trials. Figure 1b shows the schematic of a two-layer asphalt beam specimens with an old
19 pavement layer, tack coat, interlayer and HMA overlay prepared in the laboratory. The old
20 pavement layer and the new overlay can be clearly distinguished by their color (Fig.1a). The
21 two-layer asphalt beams of size 400 mm (length) \times 50 mm (width) \times 90 mm (thickness) were
22 then cut from the slabs prepared. To replicate a crack in an old distressed pavement surface

1 layer, a notch was introduced in the bottom layer of the asphalt concrete beams. Two different
2 notch depths viz. 25 mm and 40 mm, approximately accounting for 55 % and 90 % of the
3 thickness of the old pavement, were considered to investigate the influence of the crack depth
4 on the overall performance of the geosynthetic-interlayers.

5 Further, for the digital image analysis, a speckle pattern was created on the face of the
6 asphalt beams by coating the surface completely with a white paint followed by spraying a
7 black paint. The speckle pattern helps to determine the displacement and strain fields in the
8 specimens during the fatigue testing, which are otherwise difficult to obtain using the strain
9 gauges.

10 Similarly, the asphalt concrete specimens of dimensions 300 mm (length) × 300 mm
11 (width) × 90 mm (thickness) were prepared as per the procedure mentioned above for
12 interface shear strength tests with and without interlayers.

13

14 **4. TESTING PROGRAM**

15 The testing program has been divided into two stages as shown in Table 2. During the
16 first stage, pre-cracked (with 25 mm or 40 mm crack depth), two-layer asphalt beams with
17 and without geosynthetic-interlayers were investigated to estimate the crack-resisting
18 potential under flexural fatigue four point bending tests. In this test, the resistance against the
19 crack propagation into the new asphalt overlay was evaluated in terms of the total number of
20 repeated load cycles completed effectively before reaching the failure. The failure is defined
21 as a complete breakage of specimen at which, no further load is resisted by the specimen.

1 During the second stage, interface bond strength of the geosynthetic-interlayers were
2 quantified under shear and tensile mechanisms. The interface shear strength was determined
3 using an interface shear strength test, while, the interface tensile bond strength of the
4 geosynthetic-interlayers and the tack coat combination was determined using adhesion
5 tensile strength test.

6 **4.1. Flexural fatigue test:**

7 The flexural fatigue test was performed to investigate the effectiveness of two-layer
8 asphalt specimen with and without geosynthetic-interlayers under repeated loading
9 conditions as per ASTM D7460 (2010). The two-layer asphalt specimens with 25 mm and
10 40 mm deep and 10 mm wide notch were tested under a four point loading condition in a
11 load controlled mode as shown in Fig. 4. The tests were performed at normal room
12 temperature (25 °C) and repeated on duplicate specimens to verify the repeatability of the
13 data. A continuous haversine type constant load pattern with a frequency of 1 Hz was applied
14 to replicate the repeated traffic loading conditions i.e. equivalent single axle wheel contact
15 pressure of 550 kPa. To simulate the contact pressure of 550 kPa, maximum load (P) to be
16 applied on the specimen was back calculated from the maximum flexural stress equation (1)
17 derived for the case as per ASTM D7460 (2010):

$$18 \quad \sigma_f = \frac{PL}{bh^2} \quad (1)$$

19 Where, σ_f is the maximum flexural stress in MPa (= 0.550 MPa), P is the maximum load
20 applied in kN, L is the length of the beam in m, b and h are the width and thickness of the
21 beam in m. The maximum load (P) is calculated to be 0.6 kN. During the testing, a seating

1 load of 0.06 kN, which is 10 % of maximum load, and the maximum load of 0.6 kN were
2 continuously applied and the corresponding vertical deformations at mid-span of the
3 specimen were recorded. These vertical deformations were used to calculate the flexural
4 strains at the bottom most layer of the asphalt beams.

5 The corresponding flexural strains can also be calculated using equation 2:

$$6 \quad \epsilon = \frac{108\delta(h-t)}{23L^2} \quad (2)$$

7 Where, ϵ is the maximum flexural (tensile) strain in the sample, δ is the vertical deformation
8 at mid-span of the sample in mm.

9 The applied constant load (P) and the calculated strain can be used to represent the
10 flexural fatigue load-strain relationship of the two-layer asphalt beams. Note that the
11 maximum flexural stress considerably varies during the test due to propagation of the crack;
12 hence, the data is presented in terms of flexural fatigue load with strain. Figure 5 shows a
13 typical flexural fatigue load-strain curve for two-layer NG asphalt beam with 25 mm crack
14 depth. Notice that the flexural cyclic strain decreases with increasing number of load
15 repetitions. This may be due to the reduction in the rate of permanent deformations in each
16 cycle with number of load repetitions. The load-strain patterns obtained are different for
17 specimens with different crack depths and interlayer type. The flexural test results for
18 different cases are analyzed further and presented in the subsequent sections. Generally, the
19 flexural fatigue test gives an idea of either fatigue life of the specimen under repetitive
20 loading or the maximum flexural stress that can be applied on a beam. This test does not
21 provide any information on the propagation of the existing cracks or crack development and

1 delamination of the interlayers during the testing process. Generally, the beams are
2 instrumented with asphalt strain gauges to obtain localized information at critical positions.
3 This method may not provide information at all the critical locations, especially when two
4 layer laminated asphalt beams are employed. Hence, a continuous monitoring of complete
5 strain fields across the specimen surface would provide a valuable information on the crack
6 propagation, strains corresponding to the interface failure and critical tensile strains at the
7 crack tip (Kumar and Saride, 2017; Safavizadeh et al. 2015).

8 In this study, the strain field in the specimens at different load cycles was investigated
9 using a two-dimensional, full field optical technic known as digital image correlation (DIC).
10 The DIC technic is an optical data analysis method, which employs a mathematical
11 correlation technic to measure the changes in a series of images accurately. The correlation
12 between the un-deformed pattern of the reference image and the deformed pattern of the
13 images are used to obtain the two-dimensional full field displacements. The displacement
14 fields are then computed through gray level correlations between the reference image and the
15 images with deformed patterns using a commercial software (VIC-2D). The strain fields are
16 then computed from the gradients of displacement fields obtained. Figure 4 shows the area
17 on the specimen considered for the DIC analysis. The selected region was continuously
18 focused and the images were recorded at various load cycles until the failure.

19 **4.2. Interface shear strength test:**

20 A large scale interface shear test device used in the current study (Fig. 6) is in compliant
21 with ASTM D5321 (2008). The two-layer asphalt samples were prepared in the two half
22 boxes separated by an interface zone as explained in section 3.4. A constant horizontal

1 displacement rate of 1 mm/min was applied along with a constant normal load perpendicular
2 to the interface zone. The tests were repeated twice for different normal stress conditions
3 viz., 30 kPa, 60 kPa and 120 kPa at a test temperature of 25 °C. The data obtained from the
4 test in terms of shear stress and horizontal displacements were analyzed for interface shear
5 characteristics.

6 **4.3. Adhesion tensile test (ATT):**

7 The interface tensile bond strength between the tack coat and the interlayer is very critical
8 as the tack coat is found to be the weakest zone during a pavement failure due to improper
9 bonding. An adhesion tensile test (ATT) apparatus compliant with ASTM D4541 (2002) is
10 modified and fabricated in-house for the current study to determine the pull-off tensile
11 strength of the tack coat applied on the geosynthetic-interlayers. The test setup consists of
12 two 100 mm size square plates, one with an opening at the center, to accommodate the pull-
13 stub attached on to the geosynthetic material.

14 The geosynthetic-interlayers cut into 100 mm square grid size were placed in between
15 the two steel plates and clamped by mechanical bolting. A binder tack coat of about 0.08 g
16 was heated up to a temperature of 135 °C and applied on the geosynthetic material exactly at
17 the center and a pull stub of known diameter was hardly pressed against it (Ferrotti et al.
18 2012).

19 The whole assembly of pull stub and the plates were then conditioned at a test temperature
20 for 24 hours before placing the pull-stub assembly in the universal testing machine as shown
21 in Fig. 7. The ATT test was performed at three different temperatures viz. 20 °C, 30 °C and
22 40 °C as the interface tensile bond strength is highly sensitive to the temperature. A tensile

1 load was applied at a constant displacement rate of 1 mm/min until the pull-stub gets
2 completely detached from the interlayer material and the corresponding peak load was noted.
3 The pull-stub diameter and the tensile pull-off load with respect to the displacement data is
4 analyzed to obtain the interfacial tensile bond strength.

5 **5. RESULTS AND DISCUSSIONS**

6 **5.1. Flexural fatigue test results**

7 The flexural fatigue test conducted in a load controlled mode helps to investigate the
8 influence of the geosynthetic-interlayer on resisting the crack propagation in a pre-cracked
9 asphalt beam specimen. The applied cyclic flexural stress causes a continuous increase in the
10 flexural strain until the specimen does not sustain any further load cycles (N) causing a
11 complete fracture in the specimen. The results obtained from the flexural fatigue tests on two-
12 layer asphalt beams with 25 mm and 40 mm crack depths are presented in Fig. 8, which
13 depicts the variation of vertical deformations with number of load cycles (N). In both the
14 configurations, the unreinforced (NG) specimens failed to resist a large number of load
15 repetitions before the crack propagated into the overlay. However, as expected, the specimens
16 with 25 mm deep notch have sustained higher number of load repetitions than the specimens
17 with 40 mm notch depth. Contrarily, the specimens with geosynthetic-interlayers have
18 resisted vertical deformations for a large number of load repetitions. This indicates that the
19 specimens with interlayers have improved the performance of the asphalt overlays by
20 restricting the vertical deformations and crack propagation. In addition, the geosynthetic-
21 interlayer with relatively low initial stiffness (G1) has performed superior than the interlayer
22 with high initial stiffness (G3) as shown in the Figs. 3a and 3b. This could be due to the

1 smooth transfer of strain energy from the cracked pavement to the geosynthetic-interlayer,
2 which has near similar elongation properties, resulted in initiating the membrane effect in G1
3 interlayer. To mobilize this effect in the case of stiffer interlayers such as biaxial geogrid
4 specimens (G3), excessive deformations are necessary. It can be inferred from Fig. 8 that the
5 vertical deformations required for the G3 specimen are higher than the G1 specimen at all
6 load repetitions. For instance, at 100 load cycles, the vertical deformation in G1 and G3
7 specimens were respectively, 2.5 and 5.8 mm, i.e. about 66 % higher deformations are
8 required to mobilize the membrane effect in the case of stiffer geosynthetic-interlayers. The
9 other possible reason for the inferior performance of G3 interlayers could be attributed to
10 their high initial stiffness, which reduces the interfacial bonding between the pavement layers
11 (Graziani et al. 2014). However, the thermal effect on the interlayers plays an important role
12 in their performance as well (Gonzalez-Torre et al. 2014; Norambuena-Contreras et al. 2016).

13 To investigate the effectiveness of interlayers in improving the fatigue life of two-layer
14 asphalt specimens, a non-dimensional performance indicator known as improvement ratio
15 (I_R) is introduced. The improvement ratios are calculated at different vertical deformations
16 of the asphalt beams. The I_R is expressed mathematically as shown in equation 3.

$$17 \quad I_R = \frac{N_I}{N_U} \quad (3)$$

18 Where, N_I is the number of load cycles sustained by samples with interlayer before failure at
19 a given vertical deformation and N_U is the number of load cycles sustained by samples
20 without interlayer before failure at the same vertical deformation.

1 Figure 9 shows the improvement ratio for specimens with various interlayers for both test
2 configurations (25 mm crack depth and 40 mm crack depth) at different vertical
3 deformations. Though the expected vertical deformations are low on a pavement during the
4 traffic loading, higher vertical deformations are considered in this study as an academic
5 exercise. The improvement ratio has increased with an increase in the vertical deformation
6 for all the test configurations. However, the performance improvement is observed to be
7 higher in the specimens with deeper initial crack (40 mm) than the shallow initial crack (25
8 mm). Besides, the stiffness modulus of the beams with shallow cracks is generally higher
9 than the beams with deeper cracks. Hence, the former beams have sustained a higher number
10 of load repetitions than the latter ones without interlayers, thus the decrease in the
11 improvement ratio. It can be clearly seen that for the G1 interlayer specimen with 40 mm
12 crack, an improvement ratio of about 14 is observed against about 6 for 25 mm crack at 6
13 mm vertical deformation. It can also be inferred that for G1 specimen with 40 mm crack
14 depth, the improvement ratio drastically increased after 4 mm vertical deformation. At 4 mm
15 deformation, the crack would have reached the interlayer and the specimen has resisted a
16 higher number of load repetitions due to the transfer of the strain energy to the interlayer as
17 discussed earlier.

18 Based on the image analyses, the deformation fields are calculated for a different number
19 of load cycles and the variation of deformation field along the vertical direction are plotted
20 as deformation bands as shown in Fig. 10. In each color band, the vertical deformations are
21 uniform. The image analysis considers the downward deformation as negative. It can be
22 visualized that the deformation band at the mid-span of the beams show a maximum value

1 due to the maximum curvature of the beam at failure. Note that the deformation bands in the
2 NG specimen are continuous throughout the depth of the beam, indicating that the
3 deformations are uniform throughout the depth (Fig. 10). Whereas, the deformation bands
4 are discontinuous at the interface in all the specimens with geosynthetic-interlayers. The
5 discontinuities in the deformation bands are due to the presence of interlayers at the interface
6 zone, which restrains the crack propagation into the overlay and controls the vertical
7 deformation. Hence, maximum vertical deformations are observed in the bottom layer, below
8 the interlayer zone.

9 The presence of interlayer also helps to minimize the crack propagation in the vertical
10 direction by controlling the maximum tensile strain at the crack tip. The tensile strain
11 contours in all the specimens at failure are presented in Fig. 11 and it can be observed that
12 the tensile strain for the NG specimen is very high (about 30 %). Whereas, the specimens
13 with interlayer are observed to have a less tensile strain at the crack tip, due to the presence
14 of interlayers. The interlayers absorb the tensile stresses mobilized from the cracks and
15 restrict their propagation in the vertical direction. However, the cracks are observed to
16 propagate in the horizontal direction at the interface zone at ultimate fatigue life of the beams
17 (see Figs. 10 and 11). This horizontal crack propagation phenomena reduces the bondage
18 between the old and new layers at the interface and leads to delamination of pavement layers
19 at the interface zone.

20 Figure 12 shows the tensile strains obtained from the DIC analysis and the strains
21 calculated from the equation 2 for the specimens with 25 mm crack depth. The tensile strains
22 calculated are at the bottom layer of the beam, just above the crack tip as per the equation.

1 The tensile strains obtained from the image analysis are at the tip of the crack. It can be
2 observed that the strains obtained from both the analyses seem to correlate very well with
3 each other until the failure. However, at the time of failure, the tensile strains in the specimen
4 are very high at the crack tip, which are very well captured from the image analysis. The
5 tensile strain data at the failure are very high (about 30 %) in the NG specimen as shown in
6 Fig. 12(a) than the specimens with geosynthetic-interlayers as shown in the Figs. 12(b), 12(c)
7 and 12(d). The reduction in the tensile strains, as depicted from the Figs. 12 b – d, is due to
8 the presence of the interlayers, which absorbed the strains by mobilizing the membrane
9 effect. From Fig. 12, it is also very clear that the G1 specimen has performed well in
10 controlling the tensile strains and improving the fatigue life of the asphalt overlays.

11 **5.2. Interface shear strength test results**

12 As can be witnessed from the flexural fatigue tests, the life of the HMA overlays is
13 completely dependent on the relative behavior of the old and new layers at their interface.
14 The fatigue life of the overlays seem to improve considerably with the inclusion of
15 geosynthetic-interlayers; however, the bonding between the interlayer and the old and new
16 asphalt layers is a crucial factor. Hence, to address these issues and to quantify the interfacial
17 bond strength (τ_{\max}) of various interlayers sandwiched between the old and new layers,
18 interfacial shear tests were conducted.

19 The variation of interface shear stress with normal stress applied on the specimens is
20 presented in Fig. 13. The peak interfacial shear strength envelopes (τ_{\max}) can be calculated
21 using the equation 4.

1 $\tau_{\max} = C_0 + \sigma \tan \phi_p$ (4)

2 Where, C_0 is the cohesion at pure shear condition, σ is the normal stress applied and ϕ_p is the
3 peak friction angle.

4 The interface shear strength properties for various interface configurations tested at room
5 temperature (25 °C) are presented in Table 3. The cohesion component seems to be high in
6 the case of unreinforced two-layer asphalt specimens compared to the specimens with
7 geosynthetic-interlayers. Among the geosynthetic-interlayered specimens, the cohesion
8 component of G1 specimen is higher than the G3 specimen followed by G2 specimen. It is
9 also observed that the interface shearing resistance is reduced when there is no direct contact
10 between the old and the new layers, i.e. in the case of geo-jute mat interlayers. Similar
11 observations were also noticed by Brown et al. (2001); Canestrari et al. (2006) and Ferrotti
12 et al. (2012). From Fig. 13, it can be observed that the peak friction angle of G1, G2 and G3
13 interlayers are found to be about 40°, 48° and 42°, respectively. The G2 interlayer has a
14 higher friction angle owing to its surface properties compared to the other two interlayers as
15 shown in Fig. 2. The interface shear strength (τ_{\max}) of the interlayers is found to reduce with
16 respect to the control specimen (NG). On an average, the reduction is observed to be about
17 17 %, 46 % and 32 % for G1, G2 and G3 specimens, respectively.

18 From the results, it is found that the NG interface condition has the highest interfacial
19 bond strength and the inclusion of geosynthetic-interlayer at the interface reduces the bond
20 strength. The reduction in bond strength may be attributed to the delamination of the old and
21 new pavement layers due to weak bond at the interface. Hence, the bonding of the interlayers

1 with the existing old pavement and the new overlay plays a major role, which can be
2 addressed from the interlayer's pull-off or adhesion tensile strength tests.

3 **5.3. ATT results**

4 The ATT results are obtained in the form of pull-off tensile strength expressed in MPa.
5 The tests were conducted for different combinations of the geosynthetic materials with a
6 single type of tack coat material. The tests were conducted at temperatures generally
7 prevailed in the field conditions (20 °C, 30 °C and 40 °C), as the behavior of tack coat greatly
8 depends on the temperature. The tests were repeated for 5 times in each case and the results
9 are tabulated in Table 4.

10 From the current study, it is found that the bond strength increases with a decrease in the
11 temperature as shown in Fig. 14. It can also be observed that the variation in the bond strength
12 when the temperature increases from 30 °C to 40 °C is comparatively less than the variation
13 in bond strength when the temperature increases from 20 °C to 30 °C. Among the interlayers
14 tested, the G1 proved to be effective compared to the other types, due to the presence of a
15 polymer modified binder coating on the grid, which helps to maintain a strong bond with the
16 tack coat material. Although the surface of G3 interlayer is smooth, the bond strength appears
17 to be superior to G2 interlayer. The amount of tack coat required in the case of G2 interlayer
18 is comparatively higher than the other interlayer types, as it absorbs the binder tack coat due
19 to the inherent absorbing nature of the jute material. These observations suggest that the
20 tensile bond strength not only depends up on the temperature, but also on the material
21 composition of the interlayers.

1 **6. CONCLUSIONS**

2 The performance of geosynthetic-interlayers in controlling reflection cracks and
3 improving fatigue life of HMA overlays placed over pre-cracked old pavements was
4 examined. A polyester grid coated with polymer modified binder (G1), a woven geo-jute mat
5 (G2) and a biaxial polypropylene grid (G3) were adopted based on the aperture size, adhesion
6 and tensile strength properties.

7 Initially, the fatigue life of geosynthetic-interlayered asphalt specimens was evaluated
8 under a four point repeated loading condition. A DIC technic was employed to understand
9 the crack propagation mechanism and to obtain the displacement fields and tensile strains at
10 the crack tip. The flexural fatigue test results show that all the interlayers performed well in
11 retarding the crack propagation in pre-cracked two-layer asphalt specimens. The binder
12 coated polyester grid (G1) interlayer could resist the crack effectively compared to the other
13 types. This is due to the stiffness, tensile strength and adhesion properties of the material.
14 The influence of temperature on the tensile properties of G1 and G2 is negligible, whereas,
15 the ultimate tensile strength of G3 interlayer has been reduced drastically by about 29% in
16 machine direction and 37% in CMD. Besides, the high initial stiffness of G3 interlayer is
17 responsible for delamination at the interface and there by the lower performance of this
18 interlayer. The fatigue life of the asphalt specimens was higher for larger initial cracks
19 (40mm crack depth) against shorter initial cracks (25 mm). As high as fifteen fold increase
20 in fatigue life performance is noticed in the case of G1 interlayers against control specimens
21 at 6mm vertical deformation.

1 The DIC results demonstrated that the cracks have propagated quickly to the surface in
2 the controlled specimens, whereas, the specimens with geosynthetic-interlayers were
3 effective in resisting the crack propagation. Based on the tensile strain fields on the
4 specimens, the DIC technic could accurately identify the crack initiation and mobilization of
5 membrane effect.

6 Later to analyze the interface shear strength properties and pull-off tensile bond strength
7 of the interlayer, interface shear strength (ISS) tests and adhesion tensile strength tests (ATT)
8 were performed. The ISS test results confirmed that the presence of an interlayer at the
9 interface zone reduces the interface bond strength and eventually leads to debonding. The
10 interface shear strength (τ_{\max}) of the interlayers are found to reduce with respect to the control
11 specimen (NG). On an average, this reduction is observed to be about 17%, 46% and 32%
12 for G1, G2 and G3 specimens, respectively. The ATT study helps to understand the influence
13 of the temperature and material composition on the interface pull-off bond strength values.
14 The results show that the tensile bond strength for G1 interlayer type is high compared to all
15 other types, irrespective of the temperature conditions. However, higher bond strength values
16 are achieved when they are tested at a temperature of 25 °C. The ATT results also confirms
17 the ISS test results that the performance of G1 interlayer superior to the other interlayers.

18 Overall, based on the flexural fatigue and interfacial bond strength test results, the
19 interlayers performed very well in extending the fatigue life of the specimens before crack
20 propagates and/or failure of the interface bond. The G1 interlayer with a polymer modified
21 binder coating has shown a better performance in terms of higher fatigue life, interfacial bond
22 strength and potential for retarding the reflection cracking.

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1 **Table 1:** Properties of binder tack coat

S. No.	Property	Values
1	Specific Gravity	1.01
2	Ductility (cm)	100+
3	Penetration (1/10 th mm)	66
4	Viscosity, Brookfield at 60 °C (centipoise)	460
5	Softening point (°C)	52
6	Flash point (°C)	340
7	Fire point (°C)	365

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2 **Table 2:** Summary of experimental program

Stage	Test	Interface condition	Configurations
1	Four point bending test (Flexural fatigue)	NG, G1, G2 and G3	Specimens with 25 mm deep and 10 mm wide cracks
			Specimens with 40 mm deep and 10 mm wide cracks
2	Interface shear strength test	NG, G1, G2 and G3	Normal stresses: 30 kPa, 60 kPa and 120 kPa
	Adhesion tensile strength test	G1, G2 and G3	Test temperatures: 20 °C, 30 °C and 40 °C

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Table 3: Peak envelope characteristics of interface shear strength test

S. No.	Interface type	Cohesion, C_0 (kPa)	Peak friction angle, ϕ_p (degrees)
1	NG	208.26	45.87
2	G1	173	40.28
3	G2	75.10	48.07
4	G3	128.18	42.09

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2 **Table 4:** Interface bond strength results as per ATT

S. No.	Geosynthetic type	Temperature	Interface bond strength (MPa)					Mean bond strength (MPa)	SD	COV
			1	2	3	4	5			
1	G1	20 °C	3.34	3.96	3.67	2.98	3.45	3.48	0.37	0.13
		30 °C	2.48	1.96	2.16	2.86	2.54	2.40	0.35	0.12
		40 °C	2.37	1.65	1.49	2.01	1.97	1.90	0.34	0.12
2	G2	20 °C	1.87	2.34	2.02	1.68	2.15	2.01	0.25	0.06
		30 °C	1.48	1.06	1.88	1.35	1.78	1.51	0.33	0.11
		40 °C	1.25	1.38	1.62	0.86	1.01	1.22	0.30	0.09
3	G3	20 °C	2.46	2.87	2.35	1.96	3.01	2.53	0.42	0.18
		30 °C	1.7	2.43	1.64	1.36	1.58	1.74	0.41	0.16
		40 °C	1.35	0.96	1.24	1.58	1.46	1.32	0.24	0.06

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1 **Figure Captions:**

- 2 Fig. 1: a) Photograph and b) schematic of two-layer asphalt concrete beams
- 3 Fig. 2: Geosynthetic-interlayers used in the study
- 4 Fig. 3: Tensile strength of geosynthetic-interlayers used in the study
- 5 Fig. 4: Four point bending (Flexural fatigue) test apparatus
- 6 Fig. 5: Typical fatigue load-strain curve for NG sample with 25 mm deep crack
- 7 Fig. 6: Interface shear strength test apparatus
- 8 Fig. 7: ATT setup placed in universal testing machine
- 9 Fig. 8: Variation of vertical deformation with number of load cycles
- 10 Fig. 9: Improvement ratio (I_R) for different interlayers
- 11 Fig. 10: Variation of vertical deformation bands for beams with 25 mm crack depth: DIC
- 12 analysis at failure
- 13 Fig. 11: Typical DIC results for sample with 25 mm crack depth (a) Before testing, (b)
- 14 Tensile strain contours at failure
- 15 Fig. 12: Tensile strains measured from DIC analysis and test data for sample with 25 mm
- 16 crack depth (a) NG; (b) G1; (c) G2; and (d) G3
- 17 Fig. 13: Variation of interface shear stress with normal stress at a temperature of 25 °C
- 18 Fig. 14: Variation of interface tensile bond strength with temperature

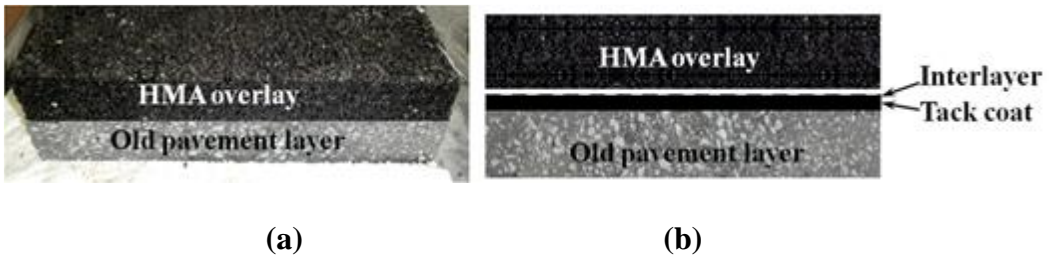


Fig. 1: a) Photograph and b) schematic of two-layer asphalt concrete beams

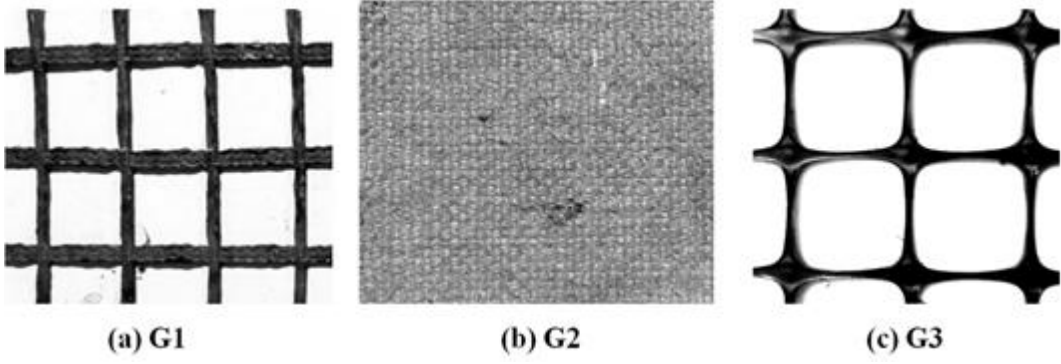


Fig. 2: Geosynthetic-interlayers used in the study

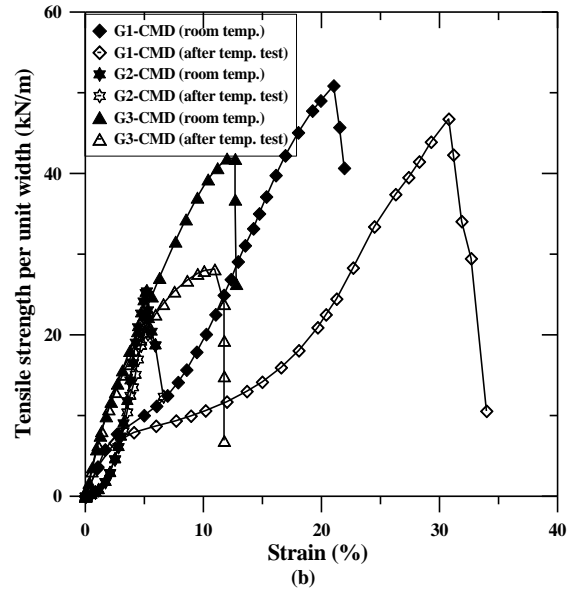
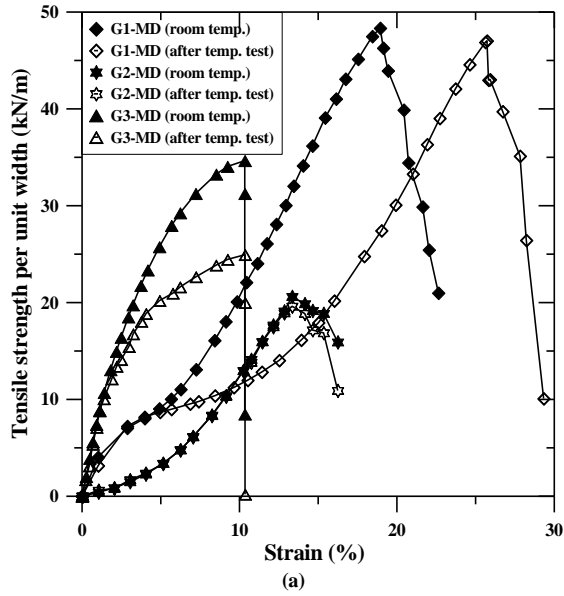


Fig. 3: Tensile strength of geosynthetic-interlayers used in the study

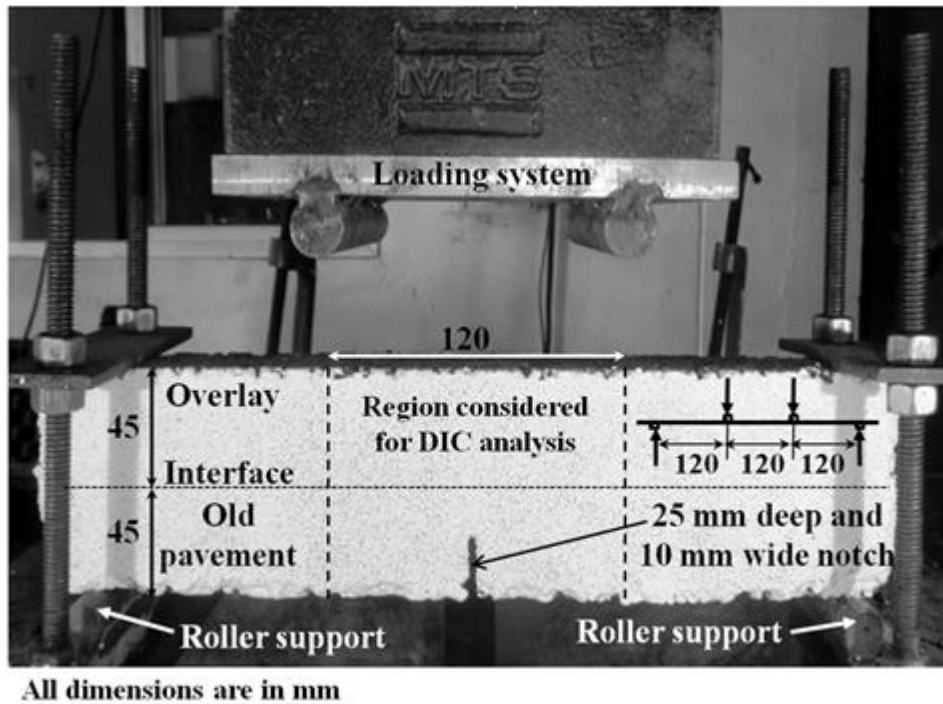


Fig. 4: Four point bending (flexural fatigue) test apparatus

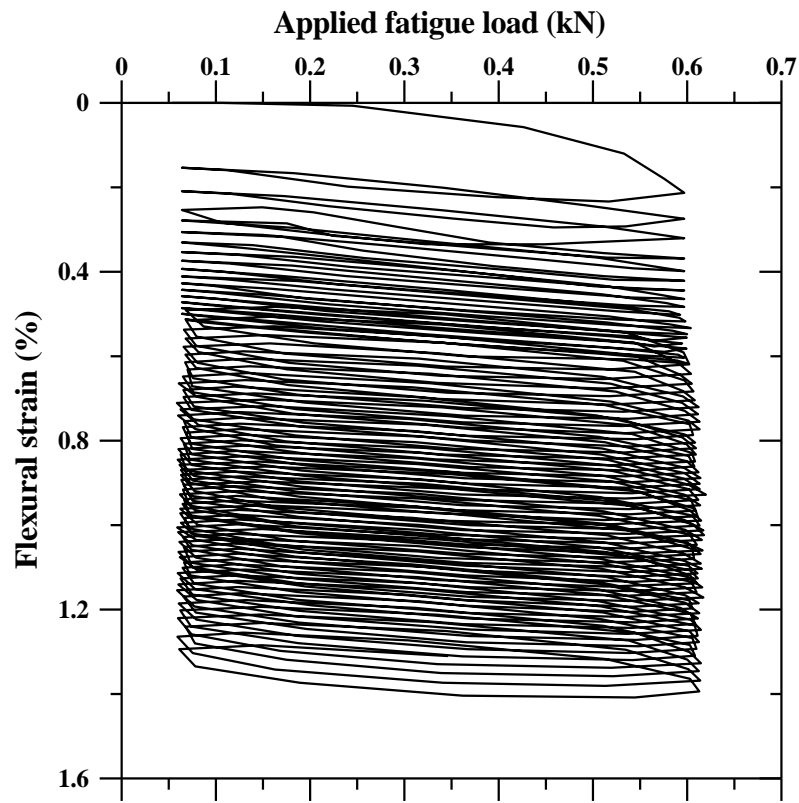


Fig. 5: Typical fatigue load-strain curve for NG specimen with 25 mm deep crack

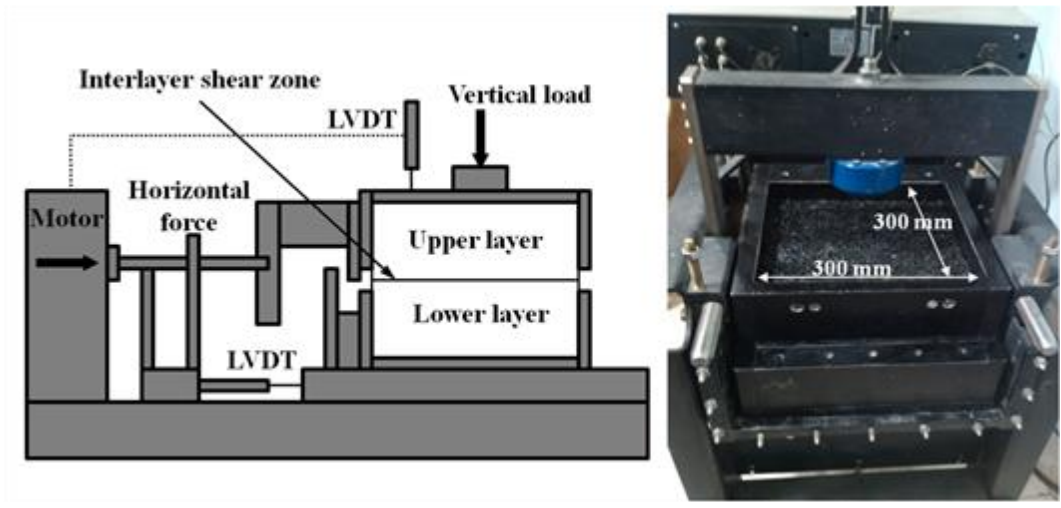


Fig. 6: Interface shear strength test apparatus

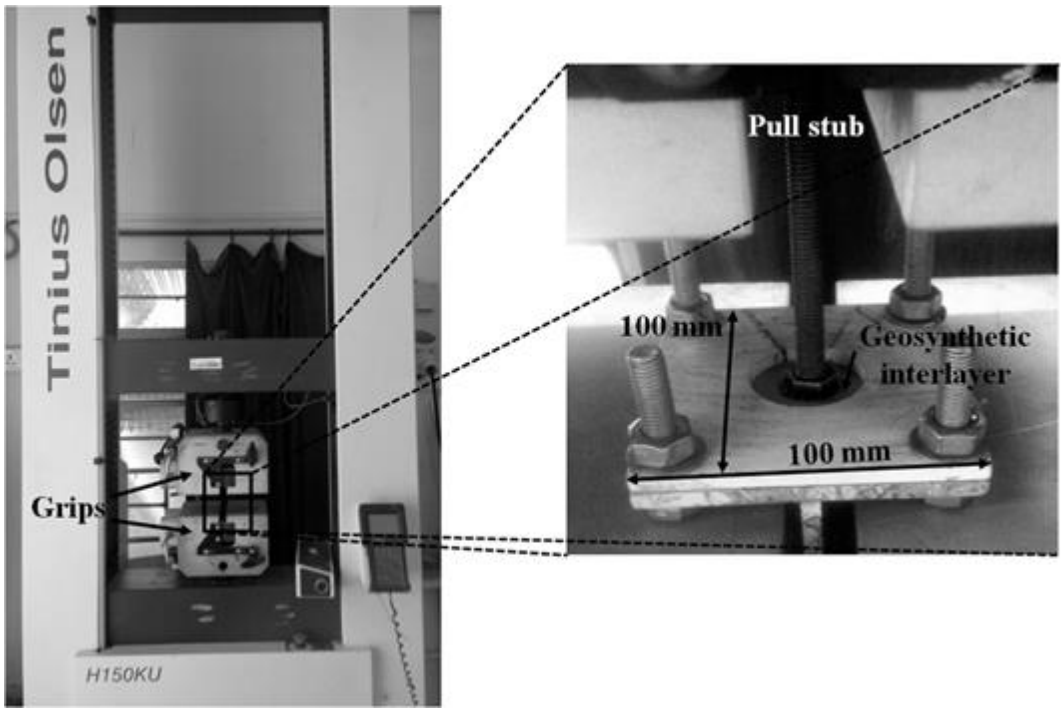


Fig. 7: ATT setup placed in universal testing machine

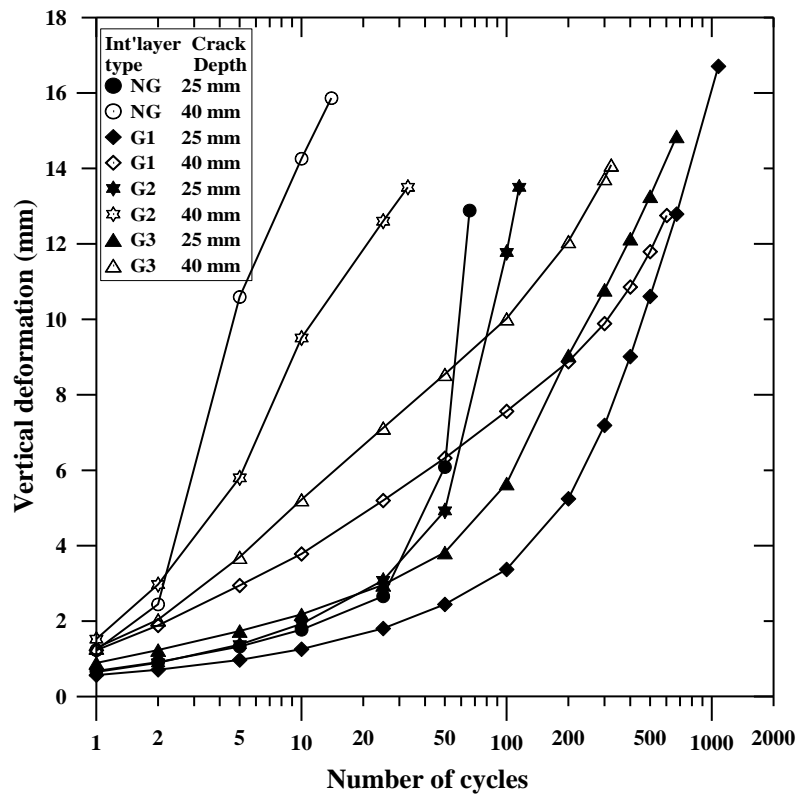


Fig. 8: Variation of vertical deformation with number of load cycles

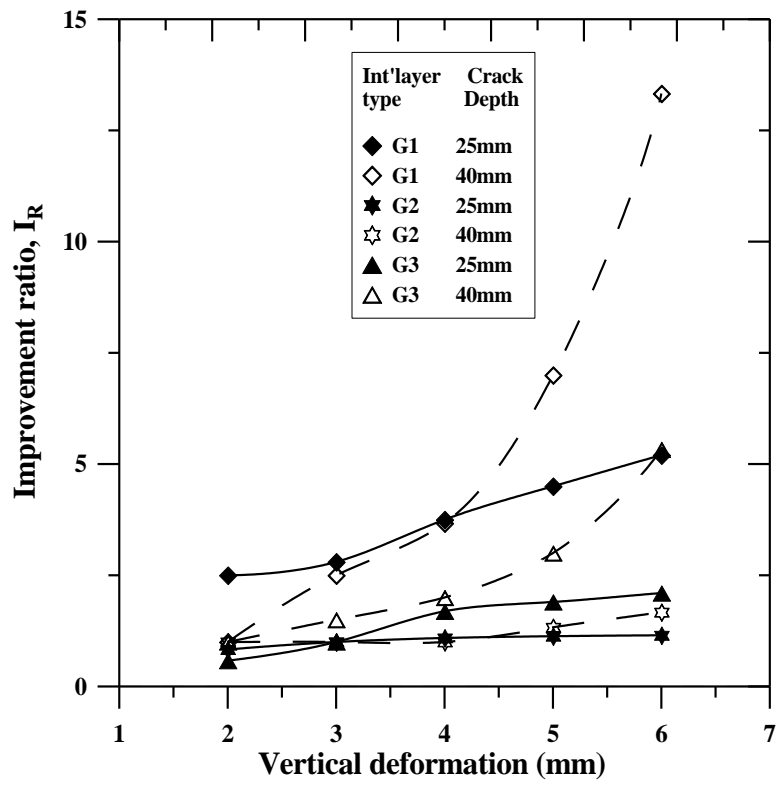


Fig. 9: Improvement ratio (I_R) for different interlayers

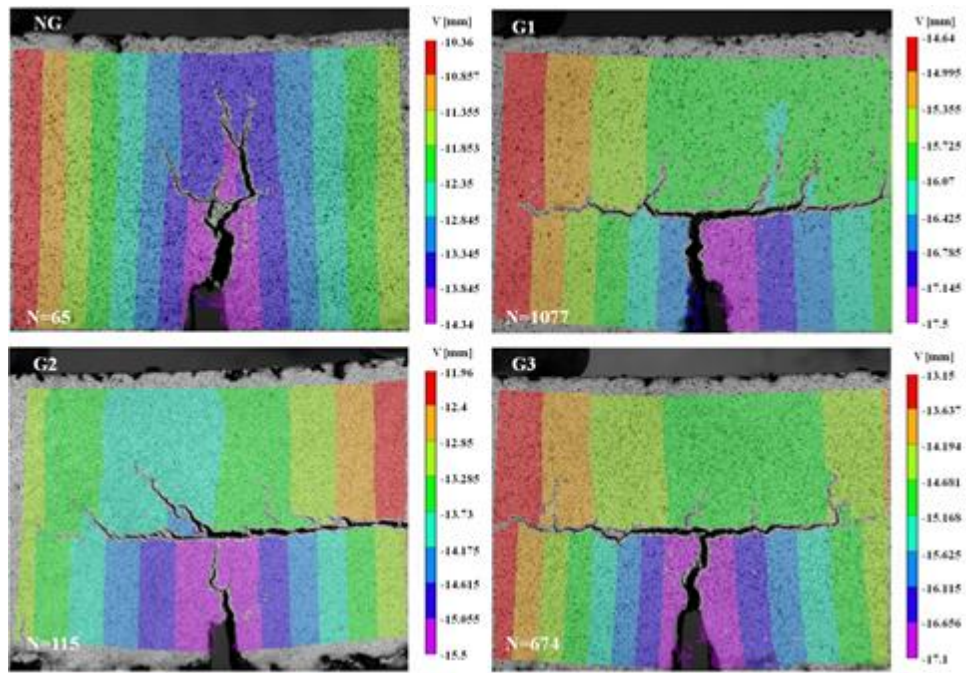


Fig. 10: Variation of vertical deformation bands for beams with 25 mm crack depth: DIC analysis at failure

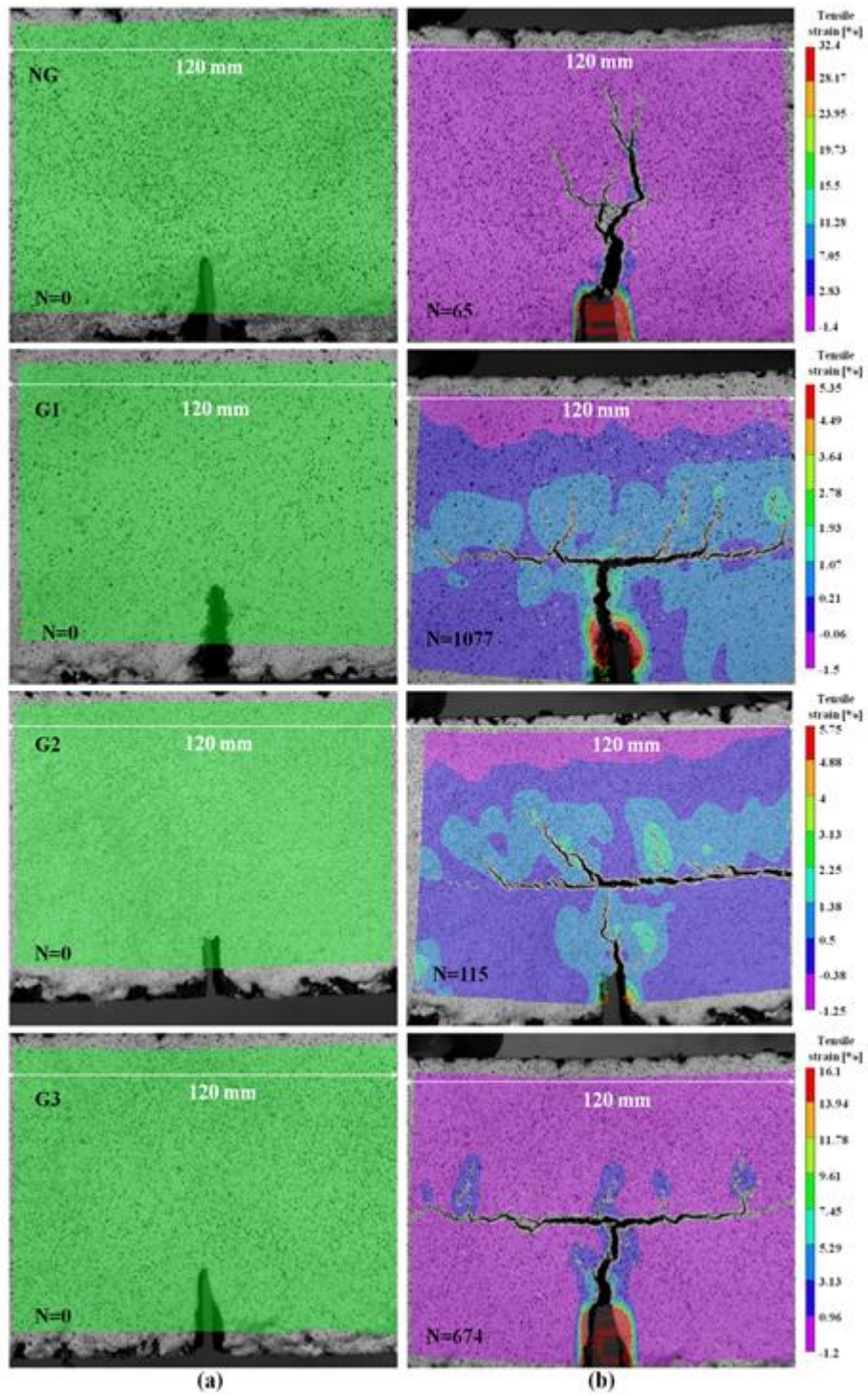


Fig. 11: Typical DIC results for samples with 25 mm crack depth (a) Before testing, (b) Tensile strain contours at failure

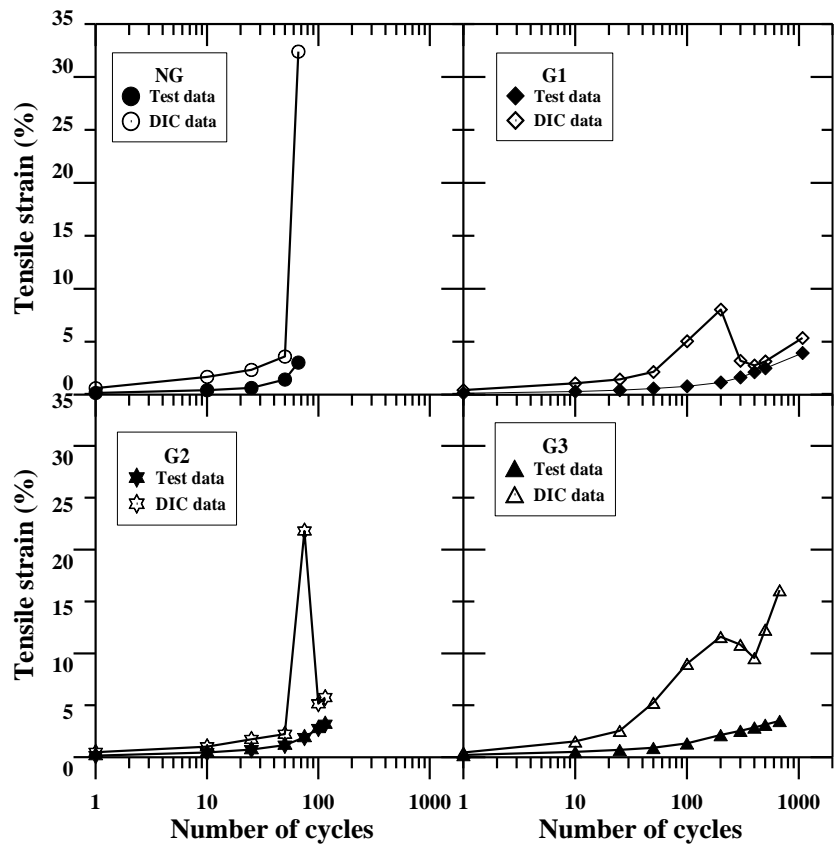


Fig. 12: Tensile strains measured from DIC analysis and test data for sample with 25 mm crack depth (a) NG; (b) G1; (c) G2; and (d) G3

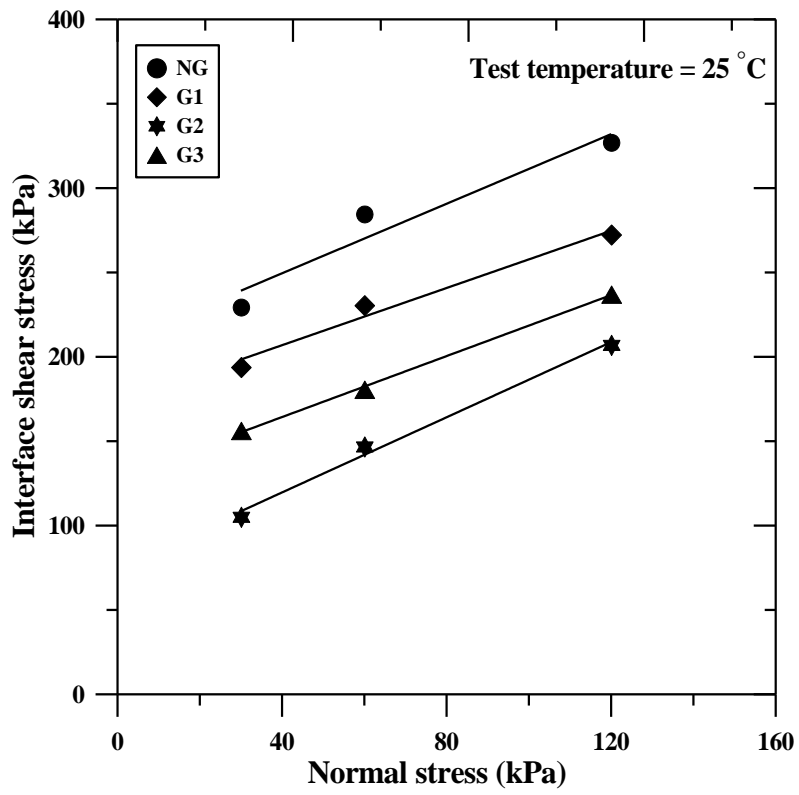


Fig. 13: Variation of interface shear stress with normal stress at a temperature of 25 °C

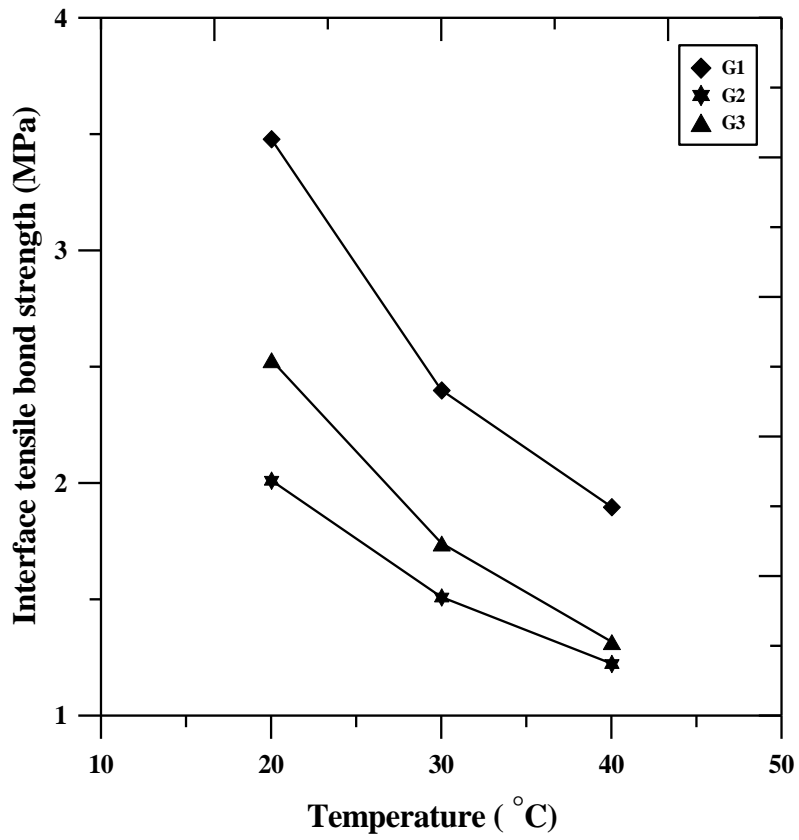


Fig. 14: Variation of interface tensile bond strength with temperature