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Abstract

This paper presents results of an investigation of the effect of geogrid reinforcement on the flexural behaviour of concrete pavements. Six concrete slab specimens having dimensions of $900 \times 900 \times 60$ mm were prepared and tested. Three of the specimens were unreinforced and taken as references. The other three were reinforced with a triaxial geogrid layer located at 40 mm from the top of the specimen. To support the specimens during the testing, two layers of recycled rubber having a side dimension of 900 mm and a 60 mm total thickness were used. According to the location of the applied load, the specimens were divided into three groups. Each group included two specimens (unreinforced and reinforced specimen). The specimens of the first group were tested by applying a load at the corner of the specimen. For the second group, the specimens were tested by applying a load at the edge of the specimen. The load was applied at the interior of the specimens for the third group. The monotonic test load was applied at a rate of 0.25 mm/min. Study parameters investigated in this study included the ductility, fracture energy and formation of cracks of the specimens. Test results obtained indicate that the flexural performance and cracking resistance of the concrete slab specimens reinforced with the geogrid can be improved.

Author Keywords: Triaxial geogrid, Portland cement concrete pavements, Corner loading,

Edge loading, Interior Loading, Bending stresses

1. Introduction

Portland cement concrete pavements are mostly reinforced with traditional steel mesh or bars to provide the strength required to resist stresses caused by traffic loads. Although steel reinforcement provides a basic strengthening for concrete pavements, there are still few limitations restraining their use (Tang et al., 2008). Such restraints include construction limitations related to placing reinforcing steel bars in thin sections such as concrete pavement overlays or resurfacing and related to chemical limitations such as concerns of steel corrosion. In addition, alternative reinforcement materials such as fibre reinforced polymer bars, tendons and grids are on the rise as alternatives to steel reinforcement (Tang et al., 2008).

Because of these limitations, several studies conducted for concrete elements reinforced with fibres such as steel, polypropylene, glass and nylon (Sorelli et al., 2006; Gupta et al., 2008; Boscato & Russo, 2009; Rajeshkumar et al., 2010; Salehian et al., 2014; Yoo et al., 2015; Karahan et al., 2016). These studies illustrated that the fibre could enhance the ductility and the cracking resistance of concrete. For the concrete pavement application, several concerns that make using the fibers as the reinforcement materials are unfeasible. For example, during mixing the concrete ingredients with the fibers, it cannot easily get a uniform mixture. Also, the workability of fiber reinforced concrete mixture still concerns for contractors, especially at high fiber volume.

Geogrid products which are considered one of the geosynthetic products are mainly used for strengthening subgrade and subbase layers beneath the pavements (Ling & Liu, 2001; Shanker & Suresha, 2006; Tang et al., 2008; Abu-Farsakh & Chen, 2011; Abu-Farsakh, Akon, & Chen, 2016). Also, several experimental and theoretical studies have been conducted on asphalt pavements reinforced with the geogrid. They include Khodaii and Fallah (2004); Doh, Baek, and Kim (2009); Sobhan and Tandon (2011); Abdesssemed, Kenai, and Bali (2015) and Zofka, Maliszewski, and Maliszewska (2016). These studies proved that the

geogrid could improve the ductile post-cracking behaviour and loading capacity of the pavements.

Geogrid products have several structural advantages making them a potential alternative of steel reinforcement for Portland cement concrete thin sections under relatively light loading conditions (Tang et al., 2008; and Arulrajah et al., 2014). These advantages include a high tensile strength and excellent chemical resistance. Also, the geogrid can provide further shear strength at the interface between the geogrid and the surrounding materials (Meski & Chehab, 2014; Siva & Agarwal, 2014, 2015a, 2015b; Wang et al. 2015; and Abdesssemed et al. 2015).

Siva and Agarwal (2014, 2015a) investigated the behaviour of prismatic and cylindrical specimens confined with one or two geogrid layers. Wang, Sheikh, and Hadi (2015) also studied the deformation capacity of cylindrical specimens confined with one, two and three polymer grid layers. They concluded that the geogrid could increase the compressive and flexural strength of concrete, along with the resistance of crack propagation which can be improved.

The failure modes in term of formation of cracks in the Portland cement concrete pavements are different in comparison with other kinds of concrete elements. The concrete pavements have a wide surface area subjected daily to the fluctuation of the traffic and environmental effects. These effects often lead to cracking of concrete. As a result, the performance of concrete pavements is eventually degraded.

In addition, the conditions of applied loads (traffic loads) and support of concrete pavements have a different impact on the failure modes created in the concrete. From these points, however, studying the behaviour of concrete pavements reinforced with the geogrid requires more investigation.

For economic reasons, the Australian market shows that the cost of manufacture and installation of the geogrid products is cheaper than the traditional reinforcement materials. There is no need for extra cost for transporting and tying the geogrid ribs like the steel bars. So, the cost of supply and placement of the geogrid materials are often lower than the cost of traditional reinforcement.

2. Objective of the study

This study on geogrid reinforcement for Portland cement concrete focuses on the flexural performance of concrete pavements through preparing six concrete slab specimens tested under a monotonic load. A single layer of triaxial geogrid placed in the concrete specimens to provide tensile reinforcement in flexure. The ductility of specimens is the main study parameter, along with formation of cracks which were monitored and evaluated.

3. Properties of triaxial geogrid

The triaxial geogrid which is one of the geosynthetic products was selected as a flexural reinforcement layer in this study. This is because their openings' shape can provide the appropriate interlocking between the geogrid layer and the surrounding concrete. In addition, the triaxial geogrid proved that the tensile and shear strengths of concrete elements reinforced with this type of geogrid were improved in comparison with other geogrid types (Meski & Chahab, 2014; Arulrajah et al., 2014).

The triaxial geogrid used in this study was made from polypropylene composite materials (Maxwell, Kim, Edil, & Benson, 2005). This product has triangular openings with a multi-directional structure. The ribs of the triaxial geogrid are connected at one point called node. The midrib depth and width of the ribs were 1.53 mm and 1.55 mm, respectively, with the length of 34 mm.

In this study, two triaxial geogrid samples, which were differed in the direction of alignment of the ribs with respect to the direction of tensile test load, were prepared and tested. They were a Machine Direction sample (Sample MD) where the geogrid ribs are parallel to the direction of the tensile test load, Figure 1 (a), and a Cross Machine Direction sample (Sample CMD) where the geogrid ribs are perpendicular to the direction of the tensile test load, Figure 1 (b).

The mechanical properties of the triaxial geogrid samples were determined using Australian Standards (AS) 3704:2005 and European Standard (EN ISO) 10319:1996. All tests were conducted at the laboratories of the School of Civil, Mining and Environmental Engineering, University of Wollongong (UOW), Australia. The Instron 500-kN tension testing machine (Model of 8033) was used to test the geogrid samples.

Two clamps made from steel plates were fabricated to clamp the samples during the testing. These clamps were fixed together with the geogrid samples using 12 bolts having a size of 13M, as shown in Figure 2.

All geogrid samples were prepared according to the EN ISO 10319:1996 Standard. The length of Sample MD, which is defined as the initial distance between two reference points located on the sample parallel to the tensile test load, was 185 mm and for Sample CMD was 130 mm. The width of Samples MD and CMD was 275 mm and 250 mm, respectively.

Table 1 and Figure 3 present the test results of the triaxial geogrid samples. For Sample MD, the maximum tensile load was 5.2 kN, and the corresponding strain was 9.8%. For the same sample, the tensile strength per unit width was 18.82 kN/m. For Sample CMD, the maximum tensile load was 6.5 kN, and the corresponding strain was 13.34%. The tensile strength per unit width was 26.11 kN/m.

Secant stiffness which is defined as the ratio of the tensile strength per unit width to the corresponding strain at a specified point on the curve was determined. The secant stiffness at 10% of strain for Sample MD was 2.62 kN/m/strain%, and for Sample CMD was 2.20 kN/m/strain%. It can be seen that there are differences in the test results of the triaxial geogrid between Samples MD and CMD. This demonstrates the importance of the placing direction of triaxial geogrid when it is applied in the field.

4. Experimental program

4.1 test matrix

Table 2 and Figure 4 present the details of the test matrix and the configurations of unreinforced and geogrid reinforced concrete specimens. Six concrete slab specimens were prepared and tested under a monotonic load. One specimen was tested for each loading position. The number of specimens selected in this study is to simulate the test conditions of study conducted by Hammons (1998). He investigated the behaviour of concrete pavements with one specimen for each case of the loading position with keeping the side dimension to thickness ratio greater than 15.

Monotonic loading was selected for testing the specimens. This is in line with findings Hammons (1998), where monotonic loading is recommended for thin sections. In addition, several experimental studies, which aimed to study the behaviour of concrete pavements under cyclic loads, were conducted with concrete specimens having a thickness of more than 150 mm (Graff et al. 2012 and Breitenbuech and Kunz 2013).

The specimens were tested at a rate of 0.25 mm/min. This rate was selected to correspond to the deflection that occur at the loaded area of a plain concrete pavement having a side dimension to thickness ratio greater than or equal to 15 and subjected to a load of 20 kN (Austroads 2004). This ratio is satisfied with the field dimensions of rigid pavements listed in

Austroads (2004) for a thickness of concrete pavements ranging between 125 to 180 mm and a side dimension between 1875 to 2700 mm. Also, the ratio is in line with conducted studies achieved by Hammons (1998) and Ruiz-Ripoll et al. (2016).

In addition, the test load process adopted here complied with the design assumptions of concrete pavements adopted by Portland Cement Association (Hammons, 1998).

According to the location of the applied load, the specimens were divided into three groups. Each group included two specimens (unreinforced and reinforced specimen). The specimens of the first group were tested by applying the load at the corner of the specimen, along the corner bisector, as shown in Figure 9 (a) and (d). For the second group, the specimens were tested by applying the load at the edge of the specimen. The load was applied at the interior of the specimens for the third group.

All specimens had the dimensions of $900 \times 900 \times 60$ mm. These dimensions were adopted to mimic as much as possible the flexural behaviour of concrete pavements and also to be suitable for testing conditions available in the laboratory.

The concrete slab with a thickness of 60 mm was selected after many calculations made using Westergaard's formulae (Haung 2004) within uncracked concrete conditions. A single tyre load of 20 kN was adopted. The field dimensions of the concrete pavement were adopted from the design requirements of rigid pavement which are reported by Austroads (2004). The dimensions of slab specimens that were selected and tested in this study were $900 \times 900 \times 60$ mm.

Three of the specimens were unreinforced and taken as references. The other three specimens were reinforced with a triaxial geogrid layer located at 40 mm from the surface of specimens. The location of the geogrid layer was selected to be within the tension zone of the slab specimen. In this study, the tension zone was below the neutral axis, which is 30 mm below

the surface. Also, providing a 20 mm concrete cover under the geogrid layer is adequate for aggregates with a maximum size of 10 mm.

Placing the geogrid at 40 mm below the surface is correct for the edge and interior loading position. For the corner loading position, the tension zone of the concrete slab generates at the upper part of the slab. Keeping the geogrid layer at same location was to simulate to what practically happens in the field. In which the geogrid reinforcement layer is placed at the same level of the rigid pavements for all kinds of loading.

The labelling of unreinforced and geogrid reinforced concrete specimens are summarised in Table 2. The first letter in the label is Unreinforced concrete specimens (U) (references) and Geogrid reinforced concrete specimens (G). The second letter in the label refers to the location of the applied load, which are: Corner, Edge, and Interior, labelled as "C", "E", and "I", respectively. For example, Specimen GC refers to the specimen which was reinforced with the triaxial geogrid and tested by applying a load at the corner of the specimen.

Two layers of recycled rubber having the dimensions of 900 mm by 900 mm with a 60 mm total thickness were used to support the specimens during the testing. This is to simulate the subbase layer. Figure 5 shows the dimensions of the recycled rubber.

4.2 Preparation of timber formworks and steel testing box

Six timber formworks were prepared in this study for moulding the concrete slab specimens, as shown in Figure 6. The inside dimensions of formworks were 900 × 900 × 60 mm.

For reinforced specimens, the walls of the formworks consisted of two parts: bottom and top part. As shown in Figure 6 (b), the height of the bottom part was 20 mm, and the height of the top part was 40 mm. The walls and the geogrid layer were fixed together using 16 bolts and screws.

A steel box was designed and fabricated in this study to support the subbase and the concrete specimens during the testing. This box was made from steel plates and had inside dimensions of $900 \times 900 \times 120$ mm. The walls of the box were made from steel plates and had a cross sectional area of 8×120 mm. The base of the testing box had the dimensions of $916 \times 916 \times 6$ mm. Two opposite walls were welded to the base. The other two opposite walls were fixed using eight 13M bolts and nuts. Figure 7 shows a photo of the steel testing box and details of its dimensions.

4.3 Casting and curing specimens

All specimens were cast on the same day using ready-mixed concrete. The concrete was mix with Ordinary Portland cement (Type general purpose), crushed stone aggregates (maximum size of 10 mm) and coarse and fine sand.

The concrete was also mixed with two kinds of supplementary cementitious materials, which were fly ash and ground granulated blast-furnace slag (GGBS). To provide adequate workability for the mix, a water-reducing admixture (WRDA-PN20) was added. The water to cement ratio adopted in this concrete mixture was 0.45. Table 3 lists details of the concrete ingredients.

A table vibrator was used to compact the concrete inside the timber formworks, with a frequency of 50 Hz. This technique had successfully achieved the purpose by providing a uniform thickness of concrete surrounding the geogrid layer. This was proved through checking the bottom surface of the reinforced specimens visually before the test and measuring the thickness of the concrete cover after the test, as shown in Figure 8.

The specimens were cured after 24 hrs from the casting and continued for 28 days. The curing process was conducted by gathering all the specimens at the same place in the laboratory.

This was to guarantee that the specimens were being exposed to the same environmental curing conditions.

During the curing process, the specimens were covered with two hessian blankets to prevent or reduce losing moisture from the concrete. In addition, two layers of plastic sheets were used to cover the specimens. This was to minimise the influences of ambient air. The specimens with the hessian covers were wetted using clean water until the curing time finished.

4.4 Preparation of specimens for testing

The configuration of the specimens prepared for testing is shown in Figure 9 (a) to (c). Linear Variable Displacement Transducers (LVDTs) were used to measure the deflection of the specimens at the specified positions, as shown in Figure 9 (d) to (f). These positions were selected to monitor the behaviour of specimens during the testing.

In total, 22 LVDTs were used. For the specimens where the load was applied at the corner, four LVDTs were installed and placed alongside the loading position. The first one (L_1) was located at a distance of 200 mm from the edge of the specimen. The remaining three LVDTs were distributed at 145 mm along the side of the specimens. In addition, two LVDTs were installed at the two other corners, at a distance of 100 mm from each edge of the specimen, as shown in Figure 9 (d).

For the specimens which were tested under the edge loading, six LVDTs were used and placed at two crossing lines, closer to the loading position. The LVDT L_4 was located at a 215 mm distance from the loading edge of the specimen. The other LVDTs were positioned at the spacing, between 120 to 155 mm, as shown in Figure 9 (e). Also, two additional LVDTs were placed at the further two corners, at a 100 mm distance from the edge of the specimen.

232 Six LVDTs were placed at a straight line close to the loading position for the third specimens
233 tested under interior loading. The LVDT L_1 was located at 120 mm from the edge of the
234 specimen. The distances between the LVDTs were 100 mm. In addition, two LVDTs were
235 placed at the other corners, at a distance of 100 mm from the specimen's edge, as shown in
236 Figure 9 (f). All LVDTs were clamped using two square hollow steel bars held by two G-
237 clamps.

238 Two layers of recycled rubber each having a thickness of 30 mm and a side dimension of 900
239 mm were used to support the concrete specimens during the testing. These layers were placed
240 inside the test box followed by the concrete slab without any bonding, as shown in Figure 10.

241 The modulus of subgrade reaction (k) of the recycled rubber was required for calculating the
242 bending stresses of the specimens. This was determined by testing a sample of recycled
243 rubber having a surface area of 152×152 mm with a thickness of 24 mm. The instron testing
244 machine (Model of 8033) was used to conduct this test. During the testing, the compressive
245 applied loads with the corresponding displacements were recorded. According to the test
246 results, the modulus of subgrade reaction of the recycled rubber was 60 MPa/m.

247 The compressive strength of concrete was determined by testing three standard cylinders
248 having a diameter of 100 mm and a height of 200 mm. These tests were carried out at the age
249 of 28 days of concrete. The average compressive strength of concrete was 29 MPa. Based on
250 AS 3600 (2009) and for the normal strength concrete, the Poisson's ratio and modulus of
251 elasticity of the concrete were assumed to be 0.2 and 25.83 GPa, respectively.

252 The flexural strength of concrete was also calculated by testing three concrete prisms having
253 the dimensions of $150 \times 150 \times 500$ mm at the age of 28 days (AS 1012.11:2000). These tests
254 showed that the flexural strength of concrete was 4.28 MPa.

During the testing, four steel strips were fixed around the perimeter of the testing box. This is to support the concrete slab. These strips had a cross section area of 3×8 mm with a length of 900 mm, as shown in Figure 9.

High strength plaster was used for capping the position of the loading of the specimen surface with a circular area of a 140 mm diameter. This material helps in providing a smooth level surface for the face of the loading piston. Thus, this ensures the stresses will be distributed uniformly.

The surface of the specimens was marked through drawing straight lines in two directions. This facilitated monitoring and evaluating the cracks that formed in the specimen. Four G-clamps were used to completely fix the steel testing box with the supports during applying the load. All the LVDTs lead wires were connected to the computer to record the deflections during the testing. A circular steel disk having a 140 mm diameter was prepared and placed between the piston of the testing machine and the specimen.

5. Experimental results and discussion

5.1 Test results

All concrete slab specimens were tested by applying a displacement controlled load at a rate of 0.25 mm/min. The 2.5 mm of surface crack width which was measured by a standard ruler edge with an accuracy of 0.5 mm was considered as the failure criterion (O'Flaherty 2005). This criterion was adopted to provide enough time for monitoring formation of cracks in the concrete specimens during the testing.

Table 4 presents the test results of unreinforced and geogrid reinforced concrete specimens. In this table, the total applied load of unreinforced specimens ($P_{ref.}$) and reinforced specimens (P_{AL}) are presented. Also, the deflections of unreinforced specimens ($\Delta_{ref.}$) and reinforced specimens (Δ_{AL}) occurred at the centre of the loaded area are listed in this table. In this study,

the positive sign was assumed when the slab is deflected downwards. While the negative sign is assumed when the slab is deflected upwards.

The increase of load-carrying capacity of the reinforced specimens compared with the unreinforced specimens was calculated using the expression of $[(P_{AL}/P_{ref.})-1] \times 100$, as shown in Table 4.

The ductility of the tested concrete specimens is one of the investigated parameters in this study. It is defined as the ability of the specimen to resist the applied load from the start of loading until a 2.5 mm surface crack width occurred. It was determined by calculating the area under the applied load versus the deflection curve (Figure 11), as listed in Table 4.

The fracture energy, which is defined as the energy required to form the microcracks in the concrete, was calculated according to Bažant and Kazemi (1990). In this study, the fracture energy was determined per slab thickness of 60 mm per unit extension of the crack length for the edge and interior loading position, as shown in Figures 14 and 17. It was determined by considering the area of the applied load versus displacement curve as the required fracture energy before cracking the concrete.

The evaluation of these findings is discussed, along with the observation of initiation and propagation of cracks.

5.2 Corner loading

Figure 11 (a) presents the curves of corner applied load versus deflection that occurred at the centre of the loaded area for Specimens UC and GC. During the load application, the maximum applied load of Specimen UC (reference) was 17 kN, as listed in Table 4. While for Specimen GC, the total load was 20 kN. This achieved an increase in the ductility for Specimen GC by about 10% more than the reference specimens.

Figure 12 shows the deflection profiles and crack map of Specimens UC and GC, along LVDTs L_1 through L_4 . The maximum deflection that occurred for Specimen UC was 8 mm at a distance of 200 mm from the edge of the specimen. While for Specimen GC, the maximum deflection was 6.5 mm at the same location.

Upward deflection took place during the testing of Specimen UC. This means that the surface level of Specimen UC lifted up higher than its original horizontal level. While for Specimen GC, the results of deflection illustrate that the original level of Specimen GC remained downwards.

According to the observations, for Specimens UC and GC, major crack propagation was extended around the corner loading position, as shown in Figure 13. After that, Specimens UC and GC failed instantly.

Finally, Specimen UC completely split into two segments. While for Specimen GC, it was still joined despite the specimen being fully cracked.

According to the test results of corner loading, the ductility and resistance of formation of the cracks were slightly improved for the specimens reinforced with geogrid.

The bending stresses generated due to the corner loading were resisted by the geogrid layer. This resulted in increasing the carrying-capacity of the geogrid reinforced concrete specimen by about 15% compared with the references.

Also, the time required to cause a 2.5 mm crack width was increased by about 10% of the total testing time for the geogrid reinforced concrete specimen. This increase reflects the feasibility of using the geogrid as a crack-formation restraining layer in concrete pavements.

5.3 Edge loading

The edge applied load versus deflection curves of Specimens UE and GE are presented in Figure 11 (b). At the surface crack width of 2.5 mm, the applied load was 21 kN for Specimen

326 UE. For Specimen GE, the total load was 24 kN. This increase achieved improvement in the
327 ductility of Specimen GE by about 12% more than Specimen UE. Also, the fracture energy of
328 Specimen GE is greater than Specimen UE by about 14%, as shown in Figure 14. Table 4
329 reports test results for Specimens UE and GE.

330 The ductility and fracture energy of the geogrid reinforced concrete specimen were improved.
331 This is due to the geogrid layer, which was embedded in the tension zone of the concrete
332 specimen, resisted the tensile stresses generated in the tension zone.

333 The improvement of flexural behaviour of the concrete specimen reinforced with the geogrid
334 comes from the layer of geogrid working as a bridge layer. This layer acts to transfer the
335 tensile stresses between both tips of the formed cracks.

336 Two structural properties in the geogrid can encourage using the geogrid as the flexural
337 reinforcement in the concrete pavements. These include the high tensile strength and being
338 adequate bonded with the surrounding concrete. As a result, the performance of concrete
339 pavements reinforced with the geogrid can be at the acceptable level during the whole of
340 design stage.

341 The deflection profiles and crack map of Specimens UE and GE are shown in Figure 15. The
342 results of the deflection were collected using two sets of LVDTs, which were L_1 , L_2 to L_3 and
343 L_1 , L_4 , L_5 to L_6 . From Figure 15 (a), the deflection of Specimens UE and GE was 2.0 mm and
344 1.5 mm, respectively, at the location of LVDT L_1 , which is located at the edge of the loading
345 of the specimen. This achieved an increase in the load resistance of Specimen GE by about
346 25% more than Specimen UE at the same location. While the deflection of Specimen UE at
347 the locations of LVDTs L_2 and L_3 was lower than of Specimen GE by about 15% more.

348 For the second set of LVDTs, from L_1 to L_6 , the deflection of specimen GE was lower than
349 Specimen UE by about 19%. Except for the deflection value, which was recorded by LVDT
350 L_5 , the deflection of Specimen GE was higher than Specimen UE by about 73%.

351 These unregulated deflection results of Specimens UE and GE, which were recorded by
352 LVDTs, illustrate that, studying the flexural behaviour of concrete pavements reinforced with
353 geogrid and tested under edge loads should be conducted by taking the effect of dowel and tie
354 bars with the adjacent slabs.

355 The failure modes of Specimens UE and GE were different as shown in Figure 16. In general,
356 the geogrid was able to delay and shift formation of cracks in the concrete. For Specimen UE,
357 two cross lines of cracks were formed and extended in the concrete (Figure 16 (a)). The first
358 one was perpendicularly extended toward the boundary of the loading position. The second
359 one was parallel to the boundary of the loading position. These crack lines finally led to
360 separating the specimen into almost four equal segments.

361 For Specimen GE, the main crack line propagated parallel to the boundary of the loading
362 position (Figure 16 (b)). After stopping the testing and lifting Specimen GE from the testing
363 box, there was another microcrack line which extended perpendicular to the main crack line.

364 The cracks in the concrete pavements are considered of one of the main defects which
365 eventually lead to deteriorate the performance of rigid pavement. During the design stage of
366 concrete pavement, the aim of the design of the concrete pavements is mainly how to keep the
367 width of cracks in the concrete tightly closed for a long time.

368 During the monitoring the test, the formation of cracks in the geogrid reinforced concrete
369 specimen took a long time, about 21%, before the 2.5 mm crack width, failure criteria, took
370 place. This increase in the testing time illustrates the feasibility of using the geogrid as a
371 crack-propagation resisting layer.

As shown in the crack map of the concrete specimen, as shown in Figure 15, a one crack line clearly formed in the concrete specimen reinforced with the geogrid compared with the two crack lines that formed in the unreinforced concrete specimen. This emphasises the role of the geogrid in improving the resistance of the concrete pavement against the formation of cracks.

5.4 Interior loading

For Specimens UI and GI, Figure 11 (c) shows the interior applied load versus deflection curves at the loaded area. Where the failure occurred (2.5 mm surface crack width), the maximum applied load was 96 kN for Specimen UI. For Specimen GI, the load was 150 kN load. This attained an increase in the ductility of Specimen GI by about 35% more than Specimen UI. The fracture energy of Specimen GI was increased by about 36% in comparison with Specimen UI (Figure 17).

For the interior loading test, the effect of geogrid reinforcement in improving the ductility and fracture energy of concrete slab specimen was clearly revealed. This is due to the high proportion of the geogrid layer embedded in the concrete slab contributed in resisting the tensile stresses. In addition, the triaxial geogrid having ribs arranged in multi-direction provides a radial tensile strength at all directions of the slab.

This is why the percentage of the improvement of the ductility and fracture energy of the geogrid reinforced concrete specimen is higher than the other kinds of loading positions.

The deflection and crack map of Specimens UI and GI monitored from LVDTs (L_1 through L_6) are shown in Figure 18. According to the reading of LVDT L_1 located at a distance of 120 mm from the edge of the specimen, the recorded deflection was 9 mm for Specimen UI and 6 mm for Specimen GI. In the same figure (Figure 18), it can be seen that the surface level of Specimen GI between LVDT L_2 and the loading position was deformed as expected. While

for Specimen UI, the surface level of the specimen lifted from its original datum by about 12 mm compared with the datum of Specimen GI.

In this loading case, for Specimen UI, the first failure led to the fracture of the concrete specimen occurred at a distance of 125 mm from the centre of the loading position. While for geogrid reinforced concrete specimen (Specimen GI), the geogrid layer acted as a bridging layer in distributing the tensile stresses and kept the specimen to deflect regularly.

Formation of cracks that occurred in this loading case can be grouped into two categories, as pictured in Figure 19. The first group of cracks was initiated and propagated radially outwards from the centre of the loaded area towards the corners and edges of the specimens. This type of cracks mostly occurred in Specimen UI, as shown in Figure 19 (a). In the second group, the cracks were extended as a curve around the load position. This type of cracks completely took place in Specimen UI and partly in Specimen GI, as shown in Figure 19 (a) and (b), respectively. At the end of the testing, few microcracks were observed in Specimen GI and radially extended towards the edges of the specimen.

According to the observations of formation of cracks in the geogrid reinforced concrete specimens, the geogrid can be an intercepting layer against the formation and propagation of cracks in the concrete. The cracks were distributed in multi-directions and within a wide area. Also, the cracks propagated by following longer paths towards the surface and depth of the specimen. As a result, the test time required to form a 2.5 mm crack width in the concrete was longer, by about 27%, in comparison with the reference specimen.

For all loading positions, the geogrid proved that it can improve the flexural behaviour of concrete pavements due to increasing the ductility and fracture energy of the pavement. Also, it can keep the serviceability index of the pavement at the acceptable level for a long time due to maintaining the width of cracks small for a long time.

The improvement of forming resistance of cracks in the geogrid reinforced concrete pavements leads to reducing the funds required for the maintenance of the concrete pavements. In addition, according to the Australian market, the cost of manufacture and installation of the geogrid is lower than the cost required for traditional reinforcement materials. As a result, the geogrid can be a feasible alternative reinforcement material for the rigid pavement applications.

6. Conclusions

Six concrete slabs, three unreinforced and these reinforced with triaxial geogrid layer, were tested under static loads at three different loading positions: corner, edge and interior of the specimen. The test results obtained illustrate that the geogrid could improve the flexural performance and cracking resistance of the concrete pavements with keeping the side dimension to thickness ratio greater than or equal to 15, more field investigations are required for concrete slabs reinforced with one or two geogrid. layers and tested under different dimension and support conditions.

1. Based on the obtained results of testing the six slabs, the following conclusions are drawn:

1. The experimental observations proved that the geogrid reinforcement could contribute to delaying the initiation and propagation of cracks in the concrete, as well as, failure modes are significantly affected by the geogrid before the failure takes place.
2. The side dimension to thickness ratio of slab specimens adopted in this study reflects the behaviour of Portland cement concrete pavements having a thickness between 125 to 180 mm and a side length between 1875 to 2700 mm.

3. The fracture energy and ductility of concrete pavements could be improved when reinforcing the concrete pavements with geogrid. So, the geogrid can be considered to improve the design rigid pavements.
4. The geogrid is characterised with high resistance to the tensile stress and corrosion. As a result, the geogrid can be a feasible alternative for steel reinforcement or they can be used as a secondary reinforcement to reduce shrinkage cracks.
5. The geogrid is manufactured from the geosynthetic materials, which are characterised with high resistance of corrosion. So, the cost of maintenance of the concrete pavements is reduced.

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Figure 19. Failure modes of interior loading. (a) Unreinforced concrete Specimen UI. (b) Reinforced concrete Specimen GI

643 Table 1. Physical and mechanical properties of triaxial geogrid.

	Property (unit)	Results	Remarks
Physical properties	Geosynthetic products	Triaxial geogrid	
	material	Polypropylene composite	
	Aperture shape	Triangular	
	Rib pitch (mm)	40 Longitudinal × 40	
		Diagonal	
	MidRib depth (mm)	1.53	measured by digital vernier calliper
	MidRib width (mm)	1.55	
	Rib length (mm)	34	
	Nodal thickness (mm)	3.94	
Mechanical properties	Nominal gauge length, MD*/CMD** (mm)	185/130	
	Specimen width, MD/CMD (mm)	275/250	
	Maximum tensile load, MD/CMD (kN)	5.2/6.5	
	Strain at maximum tensile load, MD/CMD (%)	9.80/13.34	determined using AS 3704-2005 and EN ISO 10319:1996
	Tensile strength, MD/CMD (kN/m)	18.82/26.11	
	Tensile strength at 2% strain, MD/CMD (kN/m)	7.0/7.0	
	Tensile strength at 5% strain, MD/TD (kN/m)	12.7/13.0	
	Secant stiffness (kN/m/strain %)	at 2% strain MD/CMD	
		at 5% strain MD/CMD	
		at 10% strain MD/CMD	

MD and CMD are Machine Direction and Cross Machine Direction of the triaxial geogrid, respectively.

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655 Table 2. Test matrix.

Category designation	Loading position	Specimen label	Dimensions (mm)	Reinforcement	Support
Unreinforced concrete specimens (reference)	Corner	UC	900 × 900 × 60	None	Recycled rubber
	Edge	UE		None	
	Interior	UI		None	
Geogrid reinforced concrete specimens	Corner	GC	900 × 900 × 60	Triaxial geogrid	Recycled rubber
	Edge	GE		Triaxial geogrid	
	Interior	GI		Triaxial geogrid	

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673 Table 3. Mix proportion.

Cement (kg/m ³)	Flay ash (kg/m ³)	Slag (kg/m ³)	Aggregate (10 mm size) (kg/m ³)	Coarse sand (kg/m ³)	Fine sand (kg/m ³)	Water-reducing admixture (WRDA- PN20) (l/m ³)	Water (l/m ³)	w/c
169	68	102	820	332	664	1525	153.5	0.45

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693 Table 4. Experimental results.

Specimen label	P _{ref.} (kN)	P _{AL} (kN)	Δ _{ref.} (mm)	Δ _{AL} (mm)	{(P _{AL} /P _{ref.})-1} ×100 (%)	Ductility (%)	Fracture energy (%)
UC	17.0	-----	15	-----	-----	-----	7
UE	21.0	-----	14	-----	-----	-----	12.6
UI	96.0	-----	32	-----	-----	-----	31.4
GC	-----	20	-----	14	17	10	8.2
GE	-----	24	-----	15	14	12	14.4
GI	-----	150	-----	33	56	35	49.05

P_{ref.} and P_{AL} are the total applied loads of unreinforced and geogrid reinforced concrete specimens, respectively, at a 2.5 surface crack width occurred. Δ_{ref.} and Δ_{AL} are the deflection of unreinforced and geogrid reinforced concrete specimens at the centre of the applied load, respectively, at a 2.5 surface crack width occurred.

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Figure 1. Triaxial geogrid. (a) Machine Direction (MD). (b) Cross Machine Direction (CMD).

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(a)

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(All dimensions in millimetres)

Figure 2. Testing triaxial geogrid samples. (a) The geogrid sample in the machine direction (MD). (b) The geogrid sample in the cross machine direction (CMD).

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Figure 3. Tensile loads versus strain of triaxial geogrid samples in the machine direction (MD) and cross machine direction (CMD).

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(All dimensions in millimetres)

Figure 4. Configurations of concrete slab specimens. (a) Specimen UC. (b) Specimen UE. (c) Specimen UI. (d) Specimen GC. (e) Specimen GE. (f) Specimen GI.

(All dimensions in millimetres)

Figure 5. Dimensions of the recycled rubber.

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(All dimensions in millimetres)

Figure 6. Timber formworks for (a) Unreinforced concrete specimens. (b) Geogrid reinforced concrete specimens.

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(All dimensions in millimetres)

Figure 7. Steel testing box.

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Figure 8. Checking the appearance of the concrete cover of geogrid reinforced concrete specimens. (a) Before testing. (b) After testing.

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(a)

(b)

(c)

(d)

(e)

(f)

(All dimensions in millimetres)

Figure 9. Pictures and configuration of unreinforced and geogrid reinforced concrete specimens. (a) Corner loading configuration. (b) Edge loading configuration. (c) Interior loading configuration. (d) LVDTs locations of the corner loading. (e) LVDTs locations of the edge loading. (f) LVDTs locations of the interior loading.

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(a) Recycled rubber preparation.

(b) Two recycled rubber layers beneath specimen.

(All dimensions in millimetres)

Figure 10. Recycled rubber. (a) Preparing two layers of recycled rubber. (b) Placing two layers of recycled rubber beneath the specimen.

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(a)

(b)

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Figure 11. Corner, edge, and interior applied load versus deflection at the loaded area of unreinforced and geogrid reinforced concrete specimens. (a) Specimens UC and GC. (b) Specimens UE and GE. (c) Specimens UI and GI.

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Figure 12. Deflection of Specimens UC and GC at the position of LVDTs for L₁ through L₄.

(a) Specimen UC

(b) Specimen GC

Figure 13. Failure modes of corner loading. (a) Unreinforced concrete Specimen UC. (b) Reinforced concrete Specimen GC.

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Figure 14. Fracture energy versus percentage of fracture loads for unreinforced and geogrid reinforced slab specimens, Edge loading.

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Figure 15. Deflection of Specimens UE and GE at the position of LVDTs. (a) For L₁, L₄, L₅, and L₆. (b) For L₁, L₂, and L₃.

(a) Specimen UE

(b) Specimen GE

Figure 16. Failure modes of edge loading. (a) Unreinforced concrete Specimen UE. (b) Reinforced concrete Specimen GE.

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Figure 17. Fracture energy versus percentage of fracture loads for unreinforced and geogrid reinforced slab specimens, Interior loading.

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Figure 18. Displacement of Specimens UI and GI at the position of LVDTs for L₁, L₂, L₃, L₄, L₅, and L₆.

(a) Specimen UI

(b) Specimen GI

Figure 19. Failure modes of interior loading. (a) Unreinforced concrete Specimen UI. (b) Reinforced concrete Specimen GI.

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