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Proceedings of The First MoHESR and HCED Iraqi Scholars Conference in Australasia 2017
Preface

Under the sponsorship of the Iraqi Ministry of Higher Education and Scientific Research ‘MoHESR’ and the Higher Committee for Education Development in Iraq ‘HCED’, a large number of postgraduate scholars are currently engaged in doctorate and masters degree programs in Australia and New Zealand. The scholars are from different fields of study including engineering, science, social science, law, agriculture and medicine. The First MoHESR and HCED Iraqi Scholars Conference in Australasia 2017 ‘ISCA-2017 was held at Swinburne University of Technology in Melbourne, Australia from 5 to 6 December 2017. The purpose of the conference was to celebrate the achievements of the Iraqi scholars and give them the opportunity to showcase their research outcomes to peers and the wider community of Iraqi scholars in Australia and New Zealand.

The proceedings contain four keynote and fifty general abstracts and fifty general papers. The four keynote speakers were A/Professor Salah Al-Fatlawi, Dr Daniel Mansfield, Professor Ahmed Al-Jumaily and Professor Riadh Al-Mahaidi. Reviewers, who were drawn from a specialist pool of International Advisory Committee members and other experts, peer reviewed all general abstracts and general paper submissions. The contribution of all keynote speakers and peer reviewers is greatly valued and appreciated.

ISCA2017 would not have been made possible without the support of numerous individuals and organisations. Resources provided by the host organisation Swinburne University of Technology is gratefully acknowledged. Support provided by the sponsors RMIT University, The University of Queensland and the University of New England is most appreciated. The Local Organising Committee provided extremely valuable contributions to the handling of the technical papers and the peer review process, as well as the compilation of the proceedings and book of abstracts. They also provided an excellent service in developing and maintaining the conference website. Thank you to Cristian Rojas of Red Box Communication Design who skilfully designed and produced the book of abstracts and proceedings. Thank you to Maria Han of Swinburne for providing administrative support to the conference.

Riadh Al-Mahaidi, Alaa Al-Mosawe and Mohamed Al-Younis

Editors, ISCA2017
## Committees

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Ductility of self-compacting concrete columns reinforced longitudinally with steel tubes under axial compression

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Keywords: Composite columns, self-compacting concrete, steel bar, steel tube, axial compression

This study investigates the ductility of self-compacting concrete (SCC) specimens reinforced with small diameter steel tubes in lieu of traditional steel reinforcing bars under axial compression. Five specimens were cast and tested. One specimen was reinforced with steel bars (reference specimen) and the remaining four specimens were reinforced with steel tubes. Deformed steel bars of 16 mm diameter were used in the reference specimen as longitudinal reinforcement. Steel tubes of 33.7 mm outside diameter with 2 mm wall thickness (ST33.7) and steel tubes of 26.9 mm outside diameter with 2.6 mm wall thickness (ST26.9) were used as longitudinal reinforcement in the SCC specimens reinforced with steel tubes. The nominal yield tensile strength of steel bar was 500 MPa, while the nominal yield tensile strengths of ST33.7 and ST26.9 steel tubes were 350 and 250 MPa, respectively. Although the tensile strength of the steel bars was greater than the tensile strength of steel tubes, the test results indicated that the SCC specimens reinforced with steel tubes were more efficient than the SCC specimen reinforced with steel bars. The ductility of concentrically loaded SCC specimens reinforced with steel tubes was higher than the ductility of the reference SCC specimen.

1. Introduction

Composite columns have been widely used in high-rise buildings due to their high maximum axial load, ductility, seismic resistance and fire resistance [1]. Composite columns consist of steel sections and concrete with two main types of configurations: encased steel section columns and concrete filled steel tube (CFT) columns [2]. To increase the ductility and confinement of composite columns, circular steel tube sections are used in constructing CFT columns [3]. Reinforced concrete (RC) columns traditionally use solid steel bars as longitudinal reinforcement. In this study, small diameter circular steel tubes were used as longitudinal reinforcement for RC columns. For the same cross-sectional area of a solid steel bar and a circular steel tube, the radius of gyration of the circular steel tube is higher than the radius of gyration of the solid steel bar. Filling these steel tubes with concrete can further increase the yield and ultimate strength as well as the ductility of the concrete columns under axial compression [4]. This is because the concrete inside the steel tube delays the local buckling and converts the failure mode of the steel tube wall from inward to outward buckling [5]. Since the diameter of these steel tubes is quite small, high flowability concrete is needed to completely fill the tube without segregation. The self-compacting concrete (SCC) was considered a suitable option and was used for constructing the columns in this study.
The SCC can be used in complex forms and members that contain congestion of reinforcement, without needing vibration, as it is able to compact under its own weight [6]. The SCC is very suitable for CFT columns due to the rheological properties of the SCC [7].

This study investigates the behaviour of SCC specimens reinforced with steel tubes in lieu of traditional steel reinforcing bars under axial compression.

2. Experimental Program

A total of five SCC specimens reinforced longitudinally with steel bars and tubes were cast and tested under axial compression. All specimens were 240 mm in diameter and 800 mm in height. All specimens were reinforced transversely with 10 mm round steel (R10) helices with a nominal tensile strength of 250 MPa. The first specimen was reinforced longitudinally with six deformed steel bars of 16 mm diameter having a nominal tensile strength of 500 MPa (N16). The second and third specimens were reinforced longitudinally with six steel tubes of 33.7 mm outside diameter and 2 mm in thickness having a nominal tensile strength of 350 MPa (ST33.7). The remaining fourth and fifth specimens were reinforced longitudinally with six steel tubes of 26.9 mm outside diameter and 2.6 mm in thickness having a nominal tensile strength of 250 MPa (ST26.9). The first, second and fourth specimens were reinforced transversely with 50 mm pitch steel helices (H50). The third and fifth specimens were reinforced transversely with 75 mm pitch steel helices (H75). Table 1 provides details of the column specimens tested in this study.

<table>
<thead>
<tr>
<th>No.</th>
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<td>Outside Diameter of Bars or Tubes (mm)</td>
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<tr>
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<td>6</td>
<td>16 (N16)</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>33.7</td>
</tr>
<tr>
<td>3</td>
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<td>33.7</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>26.9</td>
</tr>
<tr>
<td>5</td>
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</tr>
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2.1. Material Properties

Self-compacting concrete (SCC) was prepared in accordance with EFNARC [6] method. A number of fresh concrete mixes were investigated to achieve self-compacting concrete requirements. All the column specimens were cast on the same day using ready-mixed SCC. The average compressive strength of the SCC was 57 MPa on the 28th day.

The mechanical properties of the steel N16 deformed bars and steel R10 rounded bars were tested in accordance with AS-1391 [8]. The average tensile strengths and modulus of elasticity of three samples of N16 bars were 556 MPa and 200 GPa, respectively. The average tensile strength and modulus of elasticity of three samples of R10 bars were 400 MPa and 195 GPa, respectively.

The mechanical properties of ST33.7 and ST26.9 steel tubes were tested in accordance with ASTM A370 [9]. Full-size tubular section was used to conduct the tensile test. To avoid the crushing at the ends of the
tube due to gripping, two metal plugs fabricated from solid steel were inserted in both ends of the tube specimen. Results of testing revealed that the yield stresses of both steel tubes were not easily identified with a strain plateau and were instead determined using the 0.2% offset method. The average tensile strength and modulus of elasticity of three samples of ST33.7 were 450 MPa and 196 GPa, respectively. The average tensile strength and modulus of elasticity of three samples of ST26.9 were 355 MPa and 192 GPa, respectively.

2.2. Specimen Preparation and Testing Procedure
The PVC pipes with an inner diameter of 240 mm were used as formwork for the SCC column specimens. The PVC pipe was cut for a length of 800 mm. The longitudinal reinforcement bars and steel tubes were cut to 760 mm in order to provide 20 mm clear cover at the top and bottom of the reinforcement cage. In addition, the transverse steel helices were fabricated by coiling R10 steel bars with a 200 mm outer diameter and pitches of 50 and 75 mm. In constructing the reinforcing cages, the longitudinal reinforcement was tied with equal spacing to the inside of the helices following the reinforcement arrangement of the specimens. Figure 1 shows five fabricated reinforcing cages.

![Reinforcing cages of tested specimens](image)

The Denison 5,000 kN compression testing machine was used for testing the column specimens. Concentric axial load was applied directly through the circular steel plates attached at both ends of the column specimen. To measure the axial deformation of the column specimens, two linear variable differential transducers LVDTs were used. The LVDTs were connected directly to the loading heads of the testing machine at opposite corners. The LVDTs were connected to the data logger which recorded measurements that were saved on the control computer. All tests were carried out at a displacement controlled loading rate of 0.3 mm/min until the failure of the column specimens.

3. Ductility
To study the behaviour of SCC column specimens reinforced longitudinally with steel bars/tubes, it is important to investigate the ductility of the specimens. Ductility is an indication of the post peak axial load-
axial deformation behaviour. The more ductile column will provide a proper warning before column failure under axial load.

In this paper, the ductility was calculated as the ratio of the area under the axial load-axial deformation curve up to the ultimate deformation to the area under the curve up to the yield deformation [10]. In order to specify yield deformation, the secant line was drawn from the origin to the point of 0.75 times of the first peak load. Then, the yield deformation was specified on the ascending load-deformation curve as corresponding to the intersection point of the extension secant line and a horizontal line from the first peak load [11].

4. Experimental Results and Analysis

4.1. Failure Modes
For all specimens, initial failure of the concrete cover started with cracks after the maximum load was reached. As the axial load continued to increase the buckling of longitudinal reinforcement occurred at the mid-height of the column specimens. Finally, the specimens failed by fracture of the steel helices at the mid-height. However, specimens reinforced with steel tubes had higher axial deformations at the first fracture because the steel tubes buckled later than the steel bars. Figure 2 shows close-up view of the buckling of the longitudinal steel bar/tube and helix fracture in the tested specimens.

![Figure 2. Failure modes of the tested specimens](image)

4.2. Behaviour of SCC Specimens under Axial Compression
Table 2 reports the test results of SCC specimens tested under axial compression. Figure 3 shows the axial load-axial deformation diagrams of the five concentrically tested specimens. The axial deformation corresponding to the first helix fracture in Specimen 1 was 20 mm, while in Specimen 2 and Specimen 4 were 33.5 and 36 mm, respectively. Specimen 1 had a lower axial deformation at the first fracture because the N16 steel deformed bar buckled earlier than the steel tubes. Consequently more pressure was applied on the steel helix and caused the yielding and fracture of steel helices. Due to the tensile strengths being different, the force contribution of steel tubes ST33.7 and ST26.9 in Specimen 2 and Specimen 4 were less than the force contribution of N16 steel bars in the reference Specimen 1 by 19.9% and 37% respectively. Nevertheless, Specimen 2 had similar yield and maximum axial load as the reference Specimen 1. Specimen 4 had only 5% less maximum axial load than the reference specimen. Circular steel tubes provided more confinement to the infill concrete and increased the compressive strength of concrete, which resulted in enhancing the capacity of the columns.
The yield and maximum axial loads carried by the reference Specimen 1 were 2505 and 2734 kN, respectively. The yield and maximum axial loads sustained by the Specimen 2 were 2500 and 2729 kN, respectively, which are nearly similar to the yield and maximum axial loads sustained by reference
Specimen 1. Also, the yield and maximum axial loads carried by Specimen 4 were 2375 and 2598 kN, respectively, which are 5% less than the yield and maximum axial loads carried by reference Specimen 1.

For concentric load, the ductility of the column specimens reinforced with steel tubes were higher than the column specimens reinforced with steel bars for the same pitch of helix. The ductility of Specimen 2 and Specimen 4 were 30% higher than the reference specimen (Table 2). In spite of increasing the pitch of helix from 50 to 75 mm in Specimen 5, the ductility of Specimen 5 was 11% higher than the reference specimen. However, Specimen 3 has ductility lower than the reference Specimen by 18%. This is mainly because Specimen 3 has higher yield deformation than the reference specimen by about 13% (Table 2).

5. Conclusions

Circular steel tubes filled with Self-Compacting Concrete (SCC) were used in reinforcing the SCC column rather than using traditional steel bars. The following conclusions can be drawn.

1. As the tensile strength of steel bars and steel tubes are different, the force contribution of ST33.7 steel tubes in Specimen 2 was found to be less than the force contribution of N16 steel bars in the reference Specimen 1 by 19.9%. In spite of less force contribution of steel tubes, the column Specimen 2 had similar yield and maximum load as the reference Specimen 1 under concentric axial compression. The yield loads of the Specimens 1 and 2 were 2505 and 2500 kN, respectively. The maximum loads of the Specimens 1 and 2 were 2734 kN and 2729 kN, respectively. Hence, using steel tubes with similar tensile strength to the steel bars might result in a higher maximum load capacity.

2. For axial concentric load, specimens reinforced with ST33.7 steel tubes had maximum load capacity greater than specimens reinforced with ST26.9 steel tubes.

3. In spite of the cross sectional area of ST33.7 and ST26.9 steel tubes are the same, increasing the pitch of helices from 50 to 75 mm resulted in a higher reduction in the maximum axial load of specimens reinforced with ST26.9 tubes compared to specimens reinforced with the ST33.7 tubes.

4. The concentrically loaded SCC specimens reinforced with steel tubes showed a higher ductility than the reference specimen.

Acknowledgement

The authors thank the University of Wollongong, Australia and technical officers at the Structural Engineering laboratory. Also, the first author would like to acknowledge the Iraqi Government for the support of his PhD scholarship.

References


Experimental study of bond behaviour between NSM CFRP strips and concrete exposed to elevated temperature

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Keywords: High temperature, NSM, CFRP, Rehabilitation, Epoxy adhesive, Bond.

This paper presents the results of an experimental study of bond behavior of concrete prisms damaged by heating and subsequently repaired using NSM CFRP laminates. The near-surface mounted (NSM) technique was used in this research, where carbon fibre-reinforced polymer (CFRP) laminates were bonded using an epoxy adhesive in grooves cut into the concrete surface. Twelve heat-damaged concrete prisms were tested using a single-lap shear test. The results were compared with reference specimens without heat damage. The results demonstrate that the residual bond strength after the repair of heat-damaged concrete with CFRP strips using epoxy adhesive is 87%, 81%, and 49% for temperature exposure of one hour at 200 °C, 400 °C, and 600 °C, respectively. The corresponding values for two hours of exposure are 76%, 65% and 44%, respectively.

1. Introduction

Since one of the problems, facing buildings is exposure to high temperatures, they should be provided with sufficient structural fire resistance to withstand such circumstances, or at least to give the occupants sufficient time to escape before strength and or stability failure ensue [1]. The fire safety of RC structures largely depends on their fire resistance, which in turn depends on the combustibility and fire resistance of their main structural elements, beams and columns. Fires can occur at any time in buildings, and the safety of occupants and maintaining the integrity of the structure are of major importance [2]. The use of fibre-reinforced polymers (FRPs) in civil engineering applications has emerged over the past 20 years. Due to the main advantages of FRP, such as its durability, high strength, low weight, easy installation and the basically unlimited availability of different shapes, it has been extensively used in structural repair and strengthening [3, 4]. In the near surface mounted (NSM) method, thin grooves are cut in the surface of the concrete member. This system was first used to strengthen bridges in Europe in the early 1950s using steel reinforcement [5]. The advantages of using near-surface mounted (NSM) compared to externally-bonded (EB) FRP strengthening are that it provides protection from external environmental damage, its application does not require extensive surface preparation work and no delamination between the fibre and the concrete at the end of the test occurs, particularly in flexural members [6, 7]. The main critical issue which needs to be investigated in the application of the NSM technique is the bond properties between the FRP material used for reinforcement and the concrete surface provided by the adhesive material. Considerable research has recently been conducted into the use of NSM CFRP bars and strips, and investigations have been conducted [8, 9] on bond properties after exposure to high temperature.
The main purpose of the present research is to study the suitability and effectiveness of the CFRP NSM laminate strengthening system to repair concrete after exposure to high temperature. Many variables were encountered in this investigation, which covered the effect of high temperature on bond strength values using direct pull-out tests (single-lap shear tests) for testing after exposure to temperatures of 200, 400, 600, 800, and 1000°C for exposure periods of one and two hours.

2. Material description

2.1 Concrete
A target compressive strength of 35 MPa was specified. The concrete mix was designed according to the American mix design method ACI 211.1-94 [10]. Cylinders of dimensions 100 x 200 mm were used to obtain the concrete compressive strength according to ASTM C39-14a [11]. The average concrete splitting tensile strength was 3.56 MPa using cylinders of 100 x 200 mm dimensions according to ASTM C496-11[12]. The top and bottom surfaces of the concrete cylinders were levelled using a grinding machine to provide uniform distribution of stresses on the concrete surfaces. The following proportions by weight were used: 1 (cement); 1.67 (fine aggregate); 2.73 (coarse aggregate). The water/cement ratio was 0.57, giving a slump about 80 mm. The average compressive and splitting tensile strength of the concrete used at the age of pull-out testing was 41.5 and 4.4 MPa, respectively.

2.2 CFRP
A FRP laminate 205/2000 with unidirectional carbon fibre in smooth surface was used. The CFRP laminate was supplied by S&P Clever Reinforced Company. The dimensions of the CFRP laminate were 20 x 1.4 mm and the effective bond length was 175 mm similar to that reported by Al-Bayati and Al-Mahaidi [13]. Three specimens of CFRP laminate were tested to compare with the data provided by the manufacturer. The properties of CFRP laminate were obtained from tension tests of tensile strength, modulus of elasticity, and ultimate strain was 1950 MPa, 208 GPa and 1.15%, respectively. The laboratory values were based on the average values of three specimens of CFRP strips.

2.3 Epoxy
One adhesive was used for unheated and heat-damaged concrete prisms in this investigation. Epoxy resin adhesive was utilized for the FRP laminate system to bond the CFRP strips to the concrete substrate using the NSM method. The typical properties provided by the manufacturer of compressive strength, and flexural strength was 60 MPa, and 30 MPa, respectively.

3. Experimental program
During this research, the experimental work on the effect of heating on the behaviour of bond strength was based on 12 damaged concrete prisms cast in the laboratory and compared with references not exposed to heat. After 90 days’ age, the concrete prisms were exposed to temperature levels of 200, 400, 600, 800 and 1000 ºC and for two exposure periods of one and two hours. All the concrete prisms were heated under the ISO-834 standard time temperature curve. Fig. 1 shows the target temperature followed the ISO 834 fire curve reasonably well. After exposure to heating, the specimens were repaired with NSM laminates embedded in epoxy adhesive. All damaged concrete prisms were repaired with working grooves inside the concrete prisms, such as slits on the surface of substrate concrete. The grooves were 250 mm in length, 25 mm deep and 5 mm wide according to ACI 440.2R-08 [14]. All grooves were cleaned using air pressure to remove the dust from the surface of the concrete prisms. Epoxy resin primer with a mix ratio
of 3:2 (A-B) was then placed in the grooves to increase the adhesion between the epoxy resin and the surface of the concrete. CFRP laminate adhesive was inserted with the FRP strip in the groove to ensure concentric application of load. Next, a steel blade with a suitable thickness was used to push the adhesive into the grooves. Finally, the fibres were inserted in the centre of the slits, and wire tie spacers were used to ensure a good bond between the adhesive and the FRP laminate.

Every three days before testing, the concrete prism surfaces were cleaned and coated with white paint to clarify the propagation of cracks and make crack viewing easier. The prisms were then placed in position for testing. The rate of the load used in the test was 0.2 mm/min. All the specimens were tested using a MTS 250 universal testing machine.

![Figure 1](image-url)  
**Figure 1.** Temperature-time curves for 200, 400, 600 °C tests

### 4. Experimental results and discussion

The residual direct tensile strength values were recorded for two series, one for the reference and one for damaged concrete prisms subsequently repaired with NSM CFRP using epoxy adhesive. The groups are described in Table 1. The first letter of the specimen identification “S” means smooth CFRP strips with dimensions 1.4 x 20 mm; the second letters “E” means epoxy adhesive; the number “25” means specimens without heating; the numbers “200”, “400”, and “600” mean the temperature in degrees °C; 1H and 2H mean duration times in hours; “A” and “B” refer to specimen one and two. After exposure of the specimens to heating, the maximum bond strength was calculated by dividing the average maximum pull-out force by the contact area between the adhesive material and CFRP strips of 2 x 20 x 175 mm², where “2” is two faces of CFRP strip including the contact area between adhesive material and CFRP strips; “20” is the strip width and “175” is the bond length between the CFRP strip and the concrete substrate.

<table>
<thead>
<tr>
<th>Group</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>S25</td>
<td>Smooth CFRP strips without heating</td>
</tr>
<tr>
<td>S200</td>
<td>Smooth CFRP strips at 200 °C</td>
</tr>
<tr>
<td>S400</td>
<td>Smooth CFRP strips at 400 °C</td>
</tr>
<tr>
<td>S600</td>
<td>Smooth CFRP strips at 600 °C</td>
</tr>
<tr>
<td>E25</td>
<td>Epoxy adhesive specimens without heating</td>
</tr>
<tr>
<td>E200</td>
<td>Epoxy adhesive specimens at 200 °C</td>
</tr>
<tr>
<td>E400</td>
<td>Epoxy adhesive specimens at 400 °C</td>
</tr>
<tr>
<td>E600</td>
<td>Epoxy adhesive specimens at 600 °C</td>
</tr>
</tbody>
</table>

5. Bond strength of specimens after exposure to 200, 400, 600, 800, and 1000 °C

In the first period (one hour) of 200 °C exposure, the percentage of the residual bond strength was 87.1%. While, in the second period (two hours), the percentage of the residual bond strength was 76%. Fig. 2 illustrates the relationship between tensile force and displacement for samples damaged by heating and subsequently repaired with CFRP strip using epoxy adhesive and undamaged concrete prisms. When the temperature was raised to 400 °C, the residual bond strength value for repairs was 81% after one hour.
While, after two hours, the residual bond strength value for repairs with CFRP laminate and epoxy adhesive was 65%, as described in Table 1. It is clear that the bond strength and stiffness of concrete decrease more when the temperature is elevated to 400 °C. This means the concrete is more affected by heating under this temperature, as shown in Fig. 3. The residual bond strength of the prisms was 49% for CFRP laminate with epoxy adhesive after one-hour exposure to 600 °C. However, the residual bond strength of the prisms was 44% for two hours. Fig. 4 shows the relationship between tensile force and displacement of concrete prisms after repaired with CFRP strip and epoxy adhesive. It is observed from these results that the concrete prisms exposed to 600°C shows lower reduction in bond strength values compared with those that exposed to 200 and 400 °C, respectively, due to the reduction in stiffness of concrete at 600 °C and the micro-structural damage of concrete with increasing temperature. When the temperature was raised to 800 and 1000 °C, the mechanical properties of concrete decreased sharply and cracks were wide-spread along the concrete prism surfaces and inside the concrete cores. Therefore, the efficacy of repairing using the NSM technique is limited due to the magnitude of damage to the concrete at temperatures of 800 and 1000 °C. However, these prisms may not represent the full behaviour of the concrete in actual full-scale members, due to the propagation of cracks on the surface of the concrete prisms. Furthermore, plain concrete without reinforcement showed premature cracking before achieving the full ultimate load.

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Average Ultimate Load (kN)</th>
<th>Contact area (mm$^2$)</th>
<th>Average Bond Strength (MPa)</th>
<th>Reduction in Bond Strength</th>
<th>Maximum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE25-A</td>
<td>43.5</td>
<td>2 (20 x 175)</td>
<td>6.2</td>
<td>-</td>
<td>3.0</td>
</tr>
<tr>
<td>SE25-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.9</td>
</tr>
<tr>
<td>SE200-1H-A</td>
<td>38</td>
<td>2 (20 x 175)</td>
<td>5.4</td>
<td>12.9 %</td>
<td>2.8</td>
</tr>
<tr>
<td>SE200-1H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.4</td>
</tr>
<tr>
<td>SE200-2H-A</td>
<td>33.2</td>
<td>2 (20 x 175)</td>
<td>4.7</td>
<td>24 %</td>
<td>3.3</td>
</tr>
<tr>
<td>SE200-2H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>SE 400-1H-A</td>
<td>35</td>
<td>2 (20 x 175)</td>
<td>5</td>
<td>19 %</td>
<td>2.9</td>
</tr>
<tr>
<td>SE 400-1H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.6</td>
</tr>
<tr>
<td>SE 400-2H-A</td>
<td>29</td>
<td>2 (20 x 175)</td>
<td>4</td>
<td>35 %</td>
<td>2.8</td>
</tr>
<tr>
<td>SE 400-2H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
<tr>
<td>SE 600-1H-A</td>
<td>21.5</td>
<td>2 (20 x 175)</td>
<td>3</td>
<td>51 %</td>
<td>2.4</td>
</tr>
<tr>
<td>SE 600-1H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.7</td>
</tr>
<tr>
<td>SE 600-2H-A</td>
<td>19</td>
<td>2 (20 x 175)</td>
<td>2.7</td>
<td>56 %</td>
<td>2.7</td>
</tr>
<tr>
<td>SE 600-2H-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.3</td>
</tr>
</tbody>
</table>

S: Smooth CFRP, E: Epoxy adhesive, 25: Reference, 200, 400, 600: Temperature in degrees °C, 1 and 2: Duration times in hours.

Table 1. Details of prism specimens under temperature of 200, 400, and 600°C using epoxy adhesive with CFRP laminate.
Figure 2. Tensile force and displacement before and after exposure to 200°C for 1 and 2 h using epoxy adhesive

Figure 3. Tensile force and displacement before and after exposure to 400°C for 1 and 2 h using epoxy adhesive

Figure 4. Tensile force and displacement before and after exposure to 600°C for 1 and 2 h using epoxy adhesive
6. Conclusions

The main purposes of the current research were to investigate the effect of high temperature on bond strength of concrete, and to conduct experiments to determine how to repair the heat damaged concrete using NSM CFRP embedded with epoxy adhesive. Based on heat exposures of one and two hours, the following conclusions can be drawn:

- Repair with NSM is limited beyond 800 °C temperature exposure due to severe damage in the concrete substrate.
- Two hours of exposure to 200 °C, 400 °C, and 600 °C caused higher bond degradation than one hour of exposure, with the greatest reductions being associated with the highest temperatures.
- The specimens that exposed to different temperature levels over different periods and then repaired with CFRP strip-epoxy adhesive achieved higher bond strength than those repaired with other adhesive.

7. Acknowledgments

The scholarship support provided to the first author by The Higher Committee for Education Development in Iraq is gratefully acknowledged. The experimental work was carried out at the Smart Structures Laboratory of Swinburne University of Technology. The authors thank the staff of the Smart Structures Laboratory for their assistance.

8. References


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Behaviour of geogrid reinforced concrete pavements under elevated temperatures

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Keywords: biaxial geogrid, slab specimens, prism specimens, concrete pavements, elevated temperatures

This study aims to investigate the response of Portland cement concrete pavements reinforced with geogrids and subjected to elevated temperatures. Two groups of concrete specimens were tested in this study. The first group consisted of six concrete slab specimens having dimensions of 30 × 280 × 280 mm. Three of them were unreinforced (References). The other three were reinforced with biaxial geogrid placed at 15 mm from the surface. The second group consisted of nine concrete prism specimens having the dimensions of 75 × 75 × 280 mm. Three of them were unreinforced (References). The other six specimens were reinforced with biaxial geogrid located at 20 mm, for three specimens, and at 37.5 mm, for the other three, measured from the surface. All specimens were tested by placing them inside an oven with a temperature of 45˚ C and relative humidity of 10% for 56 days. The strains and rate of deformation were determined on the testing day of 88, 95, 99, 109 and 112 from the casting day. The findings illustrate that the geogrid reinforcement can reduce the strains of concrete pavements and also can make the concrete pavements deform at the lower rate when subjected to elevated temperatures.

1. Introduction

The average of temperatures during the last two decades has increased in the world. This is related to the global warming problem. The Portland cement concrete pavements are directly affected due to this increase. The reason is the concrete pavements have a large surface area which daily exposes to fluctuations of the environmental conditions: temperature, wind speed, and relative humidity.

In addition, the self-weight of concrete of the pavement and friction forces with the subgrade prevent the pavement from the contraction or expansion freely when subjected to elevated temperatures. As a result, thermal stresses leading to cracking of concrete are created. Thus, the durability of concrete pavements is adversely affected.

Geogrids which are considered one of the geosynthetic products are fundamentally applied to improve the behaviour of subgrade and subbase layers beneath the pavements [1, 2]. Khodaii et al. [3, 4]; Doh et al. [5] and Abdesssemed et al. [6] investigated the effect of geogrids on the performance of asphalt pavements. They reported that the load-deflection behaviour of the pavement was improved.

Recently, various geogrid products have been used as confinement and reinforcement materials of Portland cement concrete elements such as prisms and cylinders [7-9]. These studies emphasised that the compressive and flexural strength of the concrete elements could be increased.
In this study, the biaxial geogrid was used as a strain restraining layer to enhance the response of concrete pavements subjected to elevated temperatures.

2. Biaxial geogrid

Fig. 1 shows the sample of biaxial geogrid which was used in this study. The biaxial geogrid was manufactured from the polymers called polypropylene. It had square openings with the inner dimensions of 38 × 38 mm. The midrib depth and width were 2 and 3 mm, respectively, with 4 mm thickness of nodes, measured by a digital vernier.

Al-Hedad et al. [10] reported in their experimental study more details of properties of the biaxial geogrid that were conducted for five single rib samples and performed according to BS EN ISO 10319:1996 [11]. The results of tested geogrid samples showed that the tensile strength of the geogrid ranged between 1222 N to 1467 N with corresponding elongations of 57 to 109 mm, respectively. The average of tensile strength and elongation of the geogrid was 1341 N and 81 mm, respectively.

![Figure 1. Biaxial geogrid.](image)

3. Experimental study

Two groups of concrete specimens were tested in this study. The first group included testing six concrete slab specimens having side dimension of 280 mm and a thickness of 30 mm. These specimens were suggested by Al-Hedad et al. [10] to simulate as much as possible the behaviour of concrete pavements in the field. This is attained by providing a wide surface area, which is exposed to the effects of ambient conditions. Three of these specimens were unreinforced (References) and the other three were reinforced with biaxial geogrid placed at a depth of 15 mm from the surface, as shown in Fig. 2 (a).

The second group of specimens consisted of testing nine concrete prism specimens having dimensions of 75 × 75 × 280 mm. These specimens were prepared according to AS 1012.13:2015 [12], as shown in Fig. 2 (b). Three of them were unreinforced and taken as references. The other six specimens were reinforced with biaxial geogrid located at a depth of 20 mm, for three specimens, and 37.5 mm, for the other three specimens, measured from the top surface. More details of number and configuration of the specimens are reported in the Al-Hedad’s et al. study [10].

All specimens were cast on the same day using ready-mixed concrete. The concrete, had the compressive strength of 37.6 MPa and was mixed using ordinary Portland cement (164 kg/m3), Type general purpose. A 10 mm maximum aggregate size (829 kg/m3) with the find sand (1027 kg/m3) was used. Two types of supplementary cementitious materials, fly ash (66 kg/m3) and slag (98 kg/m3), were added. To provide suitable workability of mix, a water-reducing admixture, Type PN20, (1476 ml/m3) was added to the mixture during the mixing. The water to cementitious materials ratio adopted was 0.40 [10].

A high control temperature oven available in the laboratories of the School of Civil, Mining and Environmental Engineering, University of Wollongong, Australia, was used to test the specimens, as shown...
in Fig. 3 (a). The temperature of the oven was 45˚ C during the whole testing time. As shown in Fig. 3 (b), all specimens were placed inside the oven for 56 days and subjected to the required uniform temperature and the relative humidity of 10%. The test started at the age of the specimens at the age of 56 days and continued for 112 days from the casting day of the specimens.

![Figure 2. Moulds of specimens. (a) Slab specimens. (b) Prism specimens [10].](image1)

![Figure 3. Testing of specimens. (a) A high control temperature oven. (b) Specimens inside the oven.](image2)

### 4. Test results and discussion

#### 4.1 Determination of results

The strains of tested specimens were determined using the vertical comparator device had an accuracy of 0.001 mm. The test results were collected on the testing day of 88, 95, 99, 109, and 112 days. The findings listed in Table 1 are represented the average of five readings per each specimen per each group (three specimens), collected on the testing day.

For the slab specimens, the strain readings were determined in two directions. For the prism specimens, the strain readings were determined in one direction. The rate of deformation of the specimens were also calculated and evaluated in this study. All strain readings were taken outside the oven within the ambient room conditions of temperature 23 ± 3˚C and relative humidity 55 ± 5%. For each specimen, the reading of strain was taken approximately every 2 minutes.

#### 4.2 Test results of slab specimens

Fig. 4 shows the strains of unreinforced (US) and geogrid reinforced concrete slab specimens (GS) versus the testing time. The results indicate that the strains of Specimens GS were lower than Specimens US by about 6%. This is because of the role of geogrid layer in resisting the thermal stresses created due to the elevated temperature.

In addition, as shown in Fig. 5, the rate of strains of Specimens GS was lower than Specimens US during the whole testing time by about 5%.
During subjecting the concrete slab specimens to the testing elevated temperature, thermal stresses were created in the concrete. Strains due to these stresses were created and tried to deform the specimen, through expanding or contracting the specimens. This situation continued until the failure of the specimens that eventually leads to cracking of the concrete. The geogrid layer that was embedded in the concrete acted for resisting these stresses by reducing the amount of the strains. Thus, the reduction of strains of the concrete pavements will lead to maintain the durability of the Portland cement concrete pavements for a long time.

* The readings at this testing day, 56, were considered as reference readings for the comparison purpose.

<table>
<thead>
<tr>
<th>Label of specimen</th>
<th>Age of test time (days)</th>
<th>Concrete slab specimens</th>
<th>Strains (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>56*</td>
<td>88</td>
<td>95</td>
</tr>
<tr>
<td>US</td>
<td>0</td>
<td>0.21</td>
<td>0.255</td>
</tr>
<tr>
<td>GS</td>
<td>0</td>
<td>0.20</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>56*</td>
<td>88</td>
<td>95</td>
</tr>
<tr>
<td>UP</td>
<td>0</td>
<td>0.19</td>
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</tr>
<tr>
<td>GP20</td>
<td>0</td>
<td>0.19</td>
<td>0.25</td>
</tr>
<tr>
<td>GP37.5</td>
<td>0</td>
<td>0.19</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Table 1. Test results.

Figure 4. Strains of concrete slab specimens.

Figure 5. Rate of strains of slab specimens.

4.3 Test results of prism specimens

For unreinforced (UP) and geogrid reinforced concrete prism specimens (GP20 and GP37.5), the strains versus testing time are presented in Fig. 6. In general, the specimens with the prism shape did not show a clear influence of the geogrid reinforcement on the behaviour of Specimens GP20 and GP37.5 when compared with Specimens UP.
This is possible because the effective geogrid area in the prism specimens was small. So, the general trend, which highlighted the effect of the geogrid reinforcement on the behaviour of specimens, is not visible clearly.

For the rate of deformation of the prism specimens, the specimens deformed at a uniform rate during the whole testing time. Except at the testing time of 95 to 99, the rate of deformation of Specimens GP20 and GP37.5 was slightly higher than Specimens UP.

5. Conclusions

Two groups of concrete slab and prism specimens reinforced with the biaxial geogrid were tested in this study under the temperature of 45°C and relative humidity of 10% for 56 days. The test results obtained illustrate that the geogrid reinforcement could improve the resistance of the concrete pavements against the thermal stresses that are generated due to change of temperature. The following conclusions can be drawn:

1. When the temperature of ambient conditions is elevated, the geogrid reinforcement could contribute in resisting the thermal stresses generated in the Portland cement concrete pavements. Thus, the serviceability level of the concrete pavements can be sustained for a long time.

2. The concrete slab specimens with their adopted dimensions could be reflecting the role of the geogrid reinforcement on the behaviour of the concrete pavements when tested under elevated temperatures.
3. The rate of strains of the slab specimens reinforced with the geogrid was lower than the unreinforced specimens.

4. The rate of strains of the unreinforced and geogrid reinforced concrete prism specimens was achieved nearly at the same rate.

Acknowledgement

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References


The influence of UV level and exposure time on the performance of the epoxy based adhesive material

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Keywords: adhesive material, epoxy, flexural stress, fracture modes, UV exposure.

Many advanced engineering applications adopt adhesive joints primarily for assembly. The performance of the adhesively bonded joints is affected by many environmental factors including the sunlight exposure. To evaluate the resistance to the sunlight, the adhesive material is assessed through standard mechanical tests including three points bending (3PBT) test after been exposed to different sunlight (Ultra Violet- UV) exposure level and time. In this investigation, accelerating weatherometer chamber is programmed to deliver two different ultra-violet levels and three intervals of exposure to simulate modified ultra-violet dosages based on Melbourne-Australia UV level conditions. Then 3PBT conducted. Test result shows noticeable change in the load carrying capacity of the beam in addition to a ductile behaviour in the fracture modes when increasing the UV dosage and exposure time. Post-failure and surface analyses showed significant change in colour and less change in the surface roughness. It is concluded that the UV exposure time has irreversible influence on the adhesive material, while the UV level has more influence than the exposure time.

1. Introduction

The adhesive joint used widely in many advanced engineering applications. The joint behaviour extremely depending on the adhesive material performance in conjunction with the adherent’s surface profile [1]. The ability of transferring the loads from one side to other side of the joint is adhesive – propriety more than dependents to the adherent’s properties [2]. The surface quality as proven in many articles more dependants on the ability of the adhesive material to transfer the load by means of the mechanical interlocking and chemical reaction between the substrate surfaces and the adhesive material [3]. Therefore, the adhesive material properties are critical and any change even in the micro-level can produce significant changes in the adhesive bonded joint performance. Specially with the facts that the environmental conditions can change the most of the polymers as well as epoxy based adhesive materials[4]. The other important fact that the surface of the adhesive is taking most of the changes when exposed to the environmental conditions. This changes happen in thickness no more than 5-10 molecules in best cases [5]. This fact shows how sensitive to conduct the adhesive related investigations.

The sunlight influence on the polymers and epoxy based materials including adhesive materials is varying in it effects based on the material composition. It can vary from colour changing to fully change in chemical properties. Furthermore, noticeable change in the surface profile and mechanical behaviour can occur [6]. However, this does not imply that all the epoxy material has same range of response to the sunlight. It varies too depends on the level and duration of sunlight exposure. It is believed that the variation in the time for short term exposure is not the same mechanism of change for longer exposure time or when change the exposure irradiation level. In other words, the adhesive materials can have different responses to the UV level for the same intervals of exposure.
The sunlight contains different spectrum, the most damaging spectrum is the ultra-violet (UV) spectrum range. However, the UV spectrum represents only 6% of the total energy at the sea level. This range of energy is still high and approximately ranging from 240-320 MJ/m² yearly for Melbourne – Australia weather [7]. This energy is considered very high compared to other location around the world. This is mainly due to the thin ozone layer over the south-pole which permit to pass through high amount of the energy to the earth.

To understand the influence of the UV level and exposure time, the adhesive materials mechanically tested by three points bending test. Load-deflection relationship obtained and used for extrapolation the parameters changes impact on the alteration of the load-deflection relationship.

2. Test setup

2.1. The material and three points bending test.
The samples are made of spabond 340 epoxy based adhesive material (Gurit company). This adhesive material manufactured for bonding the CFRP with other metal components for car, marine and aviation applications. The adhesive materials come in two parts (epoxy and hardener), the mix-ratio is 2:1 by weight or volume with 5% allowed variation. Plastic moulds used to prepare the samples then cured in electrical furnace between 65°C to 75°C for 8 hr based on the manufacturer recommendations. later, the samples shaped back and polished to the desired dimensions as in (Fig.1) based on ASTM D790 – 15 [8].

![Figure 1. Sample dimensions and three-points bending test setup](image)

Then the samples placed on the Instron 10 kn platform to conduct the flexural test at two constant test rates of 0.1 and 0.5 mm/ min as shown in (Fig.1) where the span is 30 mm.

2.2. UV- exposure procedure

The weatherometer chamber model of UVTes™ shown in (Fig. 2) was used to deliver controlled UV dosage within controlled temperature. The dosage was designed based on the Victoria state - Australia yearly UV condition [7]. Two dosages as shown in table (Table.1) represent two different weather conditions were implemented for the duration of 0 hr, 40 hr, 80 hr and 160 hr.
Figure 2. The weatherometer chamber and samples settings.

<table>
<thead>
<tr>
<th>Dosage No.</th>
<th>Irradiation</th>
<th>Wavelength</th>
<th>Type</th>
<th>Source</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5 w/m²/nm</td>
<td>340 nm</td>
<td>340 - A</td>
<td>8 Florescent lamps</td>
<td>40°C</td>
</tr>
<tr>
<td>2</td>
<td>1 w/m²/nm</td>
<td>340 nm</td>
<td>340 - A</td>
<td>8 Florescent lamps</td>
<td>40°C</td>
</tr>
</tbody>
</table>

Table 1. The Ultra-Violet dosages settings

3. Results and discussions

3.1. Weight loss calculations

weight reduction observed after the samples exposed to the different UV intervals. The weight reduction can affect the material initial mechanical properties. The weight reduction calculated as a percentage of the initial weight as in the (Eq. 1) [9].

\[
\text{Weight loss} = \frac{(\text{weight before irradiated} - \text{weight after irradiated})}{\text{area of irradiated surface}} \times 100\%
\]  

Weight reduction percentage for different UV exposure intervals are shown in (Fig. 3). the percentage of weight loss for the first dosage recorded at after 40 hr of exposure and was 0.3 % and 0.88% of initial weigh then gradually increase to record 0.7 % and 1.4 % of the initial weight after 80 hr and 1% and 1.4% after 160 hr for dosage 1 and dosage 2 respectively. The relationship it might different depends on the UV source, distance and the material properties. Moreover, the relationship may vary with extended the exposure time as short-term exposure behaviour is totally different from long term exposure impact on the adhesive material.

Figure 3. Weight reduction as function to UV exposure for two different UV levels.
3.2. Three points bending tests results

3.2.1. Strain rate and curing time

The strain rate found to have noticeable influence on the adhesive material ability to hold the load as shown in (Fig. 4). When the curing time was 24 hr only, there is 34% load increase recorded by changing the strain rate from 0.5 mm/min to 0.1 mm/min while the deflection is reduced. However, 7.6 % only as increase in the load recorded by extending the curing time for one week, while slight decrease in the deflection is observed. Meanwhile, curing condition have significant increase on the adhesive material loading ability as shown in (Fig. 4) as there is about 35% load capacity increase from 1 day to 7 day curing time for the same strain rate of 0.5 mm/min and about 16% load increasing only for 0.1 mm/min strain rate. This shows that the extending the curing condition for more than one week can enhance the load carrying capacity and bring more stability to the deflection response and eventually on the fracture mode.

![Graph showing strain rate and curing time influence on flexural load capacity.](image)

**Figure 4.** The strain rate and curing time influence on the flexural load capacity.

3.2.2. UV influence on the load-deflection relationship.

The three points bending test (3PBT) results in term of load – deflection relationships are blotted as functions to different UV exposure duration in (Fig.5). The results show that there is high tendency to shows lower load carrying capacity by increasing the UV exposure. For example, in the first dosage results, there are approximately 7.3 %, 14.2%, and 35% drop in the load capacity after 40 hr, 80 hr and 160 hr respectively of exposure time. While, more ductile fracture modes are observed by increasing the exposure time. This is allies with the general conclusion made by [10].

From other hand, increasing the dosage irradiation level from (0.5 w/m²/nm) to (1 w/m²/nm) shifts the adhesive material trend to show generally ductile behaviour. While noticeable reduction in the loading capacity were recorded as shown in (Fig.5). This behaviour as a reaction to the UV exposure can explain the significance to provide suitable protection design for the adhesively bonded joint specially made of epoxy based adhesive materials[11].

![Graph showing UV influence on load-deflection relationship.](image)
3.3. Surface inspection

The UV exposure have different influence on many materials [11]. Colour changes one of the indicators that can be linked to the mechanical changes and surface properties transformation. In many cases, colour stability and clarity are very critical to decide what type of adhesive material to be used in certain application. Therefore, it is very important to understand the UV impact on the colour change for epoxy based materials in term of Yellowing index. Linking the colour change to the UV exposure cannot be accurately correlated to the real-life exposure conditions. However, it is still successful approach for studying the adhesive materials behaviour for accelerated aging procedures. A Minolta shading examination spectrometer is used to identify the change from the unexposed condition, and ongoing with layout gives an indication of how the colour change take place with extending the UV exposure time. This model is proposed based on the ASTM D1923 and ASTM E313 [12] as shown in (Fig.6). It can easily notice the colour change for the second dosage of (1 w/m²/nm) which it starts in the ranges between 7.3-15 at the baseline up to 45 at 160 hr. Colour changes is not the only observation, the airgaps and voids at the adhesive material surface also changed at the micro-level. Further investigations is still required to address the impact of these changes on the crack- initiation and propagation.

![Figure 6](image)

**Figure 6.** Minolta shading spectrometer to categorize the UV influence on the colour changes of the adhesive material.

3.4. Fracture inspection

The transformation in the fracture mode can indicate the influence of certain parameter (s) on the material chemical and mechanical properties [13]. In the current investigation, the UV exposure of (1 w/m²/nm irradiation level) duration shifts the fracture mode from the mostly brittle, one straight line and a wide range of deformation is visible away from the basic fracture line of the baseline condition to more ductile, zig-zag multiple lines fracture and less deformation range mode after 160 hr of UV exposure (Fig.7). Between the baseline condition and 160 hr of UV exposure, the fracture transformation can be described
as linear trend. The same pattern was noticed when the irradiation level was 0.5 w/m²/nm with less trend generally.

This changes on the fracture parameters such as formation, direction and patterns can be used to show the alteration due to many different environmental or loading conditions. This transformation can be linked to the alteration in the chemical characteristics of the adhesive material due to photooxidation and the facial chemical reactions. It may see that the surface show slight change to brittle behaviour, However, the bulk of the adhesive material acts as more ductile due to the re-distribution of residual stresses due to accelerated curing procedure or thermal chemical stresses[14].

![Figure 7. Fracture mode transformation for different UV exposure durations for the second irradiation level of (1 w/m²/nm)](image_url)

**Conclusions**

The main conclusions can be listed as following:

1. The weight reduction is noticeable phenomenon during UV investigations and cannot neglected even if it was in very small percentage.
2. The surface changes can be used as a reference to the chemical reactions that may lead to extensive change in the mechanical properties
3. Loading capacity reduced by increasing the exposure time and increasing the irradiation level. However, the results show more response to irradiation level than increasing the exposure time.
4. The fracture mode highly affected by the irradiation level rather than only by the exposure time, ductile behaviour was presented and this do not imply that the adhesive material will act in the same way if thinner layer is used in real-life applications.

**Acknowledgment**

The authors would like to acknowledge the technical staff in the school of engineering and carbon nexus in Deakin University- Australia for the outstanding support as well as Iraqi Ministry of Higher Education and Scientific Research to funding M. Khaleel’s researches.
Reference


Using the NSM FRP Technique with Mortar and Different Configurations for the Torsional Strengthening of RC Beams

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Keywords: Torsion, CFRP laminate, NSM, concrete beam, mortar, U-wrapping.

The near-surface mounted (NSM) FRP and the externally bonded FRP strengthening techniques were utilized to strengthen the RC beams. The externally bonded technique is extensively researched over the last two decades, however, the near surface mounted has obtained popularity in recent years due to the researchers’ suggestions as a more effective system. Flexural and shear applications using NSM technique has attracted many researchers. However, a little investigation for torsional strengthening has been made. The experimental program of the present study investigates the torsional strengthening of RC beams using the NSM technique and mortar. Two different torsional strengthening configurations were examined i.e., full wrapping and u-wrapping. Five specimens were tested using a special torsional rig. The results of the torsional capacity with respect to the angle of twist were presented. The torsional capacities of the different strengthening configurations were compared to the control specimens’ capacities. Observable increases to the torsional strength of the concrete members were obtained using different configurations of NSM technique with mortar.

1. Introduction

In addition to flexure and shear, bridge elements and many structural elements could be subjected to a significant torsional loading. Factors such as increased service loading, structural damage or deterioration, and more stringent updates in design codes could be the necessity of the needing to the torsional strengthening and rehabilitation. Many torsional strengthening techniques introduced, such as epoxy repair, steel jacketing, section enlargement and using fibre reinforced polymer (FRP) composites. The use of FRP in recent years has become the easiest and the most realistic technique to strengthen RC beams due to its high tensile strength, light weight, ease of installation and durability (Fib Bulletin 14 2001 [1]). Two techniques have been used to apply FRP materials to RC structures i.e. externally-bonded (EBR) FRP, where FRP is applied externally to the concrete surface, and near-surface mounted (NSM) techniques, where the FRP is embedded within grooves cut into the concrete cover zone (Al-Bayati and al-mahaidi 2014[2]).

EBR FRP composites have been effectively used with many strengthening configurations for upgrading the torsional deficiency of RC beams. It is seldom possible in practice to fully wrap the beam cross-section due to the presence of either a slab or a flange. Extensive research has investigated rectangular sections fully wrapped with FRP while a few studies have considered other types of configurations, such as U-wrapping and side-bonded systems. All configurations can be either continuous or in the form of discrete strips at a certain spacing (Hii and Al-Mahaidi 2006a and 2006b [3,4]; Ameli, and Ronagh 2007 [5]). However, apart from the initial study conducted by the authors (Al-Bayati et al. 2016 (6)), to date, no study has been published on NSM-strengthened beams in torsion to the authors’ knowledge. However, many studies have
been conducted to evaluate the shear strengthening of RC beams using NSM technique. The side-bonded technique has been mainly used in these studies (Barros and Dias 2006 [7], El-Hacha and Wagner 2009 [8] and Lim 2010 [9]).

Serious problems have been attracted attention toward the use of epoxy with FRP applications including hazardous toxic fumes, moisture impermeability and flammability, and the sensitivity of epoxy to temperatures higher than the glass transition (Tg) of 60-70 °C (fib-Bulletin 14 2001 [1]). The work of Wiberg 2003 [10] was one of the first attempts to use cement-based adhesive for EBR FRP beams with cementitious mortar. Recently, good bond properties have been achieved for the EBR FRP strengthening technique using a new cement-based adhesive developed by Hashimi and Al-Mahaidi 2010 [11]. In recent times, the use of cement-based adhesive with the NSM technique has been reported by Al-Abdwais 2015 [12]. The cement-based adhesive achieved by Hashimi and Al-Mahaidi 2010 [11] has low viscosity, which makes it difficult to apply in the slits used in the NSM technique. The improvement of a new cement-based adhesive was therefore necessary for NSM application. In Al-Abdwais 2015 [12] study, the researcher achieved excellent bond properties by adding polymer to the adhesive mortar, which increases the viscosity of the adhesive, and a significant ductile behaviour was observed for all specimens. The modification of the adhesive obviates dropping away and it works more functionally in practical applications. It is recommended to use the modified mortar within 30 minutes of pot-life. The results of the strengthening with NSM using modified cement adhesive were compared with the corresponding EBR test results of Hashimi and Al-Mahaidi 2010 [11]. This comparison showed that NSM was 2.5 times more effective than EBR. The flexural strengthening was then the first application of the NSM CFRP technique with modified cement-based adhesive on RC beams at the normal and high temperatures achieved by Al-Abdwais 2015 [12]. The failure of the beams at high temperature occurred at 830 C° using modified cement-based adhesive while the epoxy was limited to its transition temperature (Tg). Similar to the flexural behaviour, it is important to trial its use to improve the torsional behaviour of RC beams.

This paper is the one of the first studies carried out by the authors which evaluates the effectiveness of the NSM technique to increase torsional capacity using cement based adhesive with different CFRP laminate configurations. Two control beams and three beams strengthened for torsion with CFRP laminate using the NSM technique and mortar were prepared and tested. One beam was strengthened at all faces while the rest two strengthened beams were reinforced with u-shaped FRP laminate (three face strengthening to examine the effectiveness of the different configurations.

2. Experimental program

2.1. Beams, details and torsional strengthening technique

Five rectangular RC beams, 260 mm deep x 140 mm wide x 2000 mm long were poured. The central 1.2 m part in each beam was designed to be deficient in torsion, which caused the beams to torsionally fail. About one and half spiral cracks can be formed in this part with 45° to the beam longitudinal axis. Fig.1 (a) presents the adopted beam cross-section dimensions and the steel reinforcement details for the beams. Grooves were cut in the beams’ 20 mm cover with 200 mm spacing between the grooves for all strengthened beams, which represents 0.75 of the beam depth (260 mm). In this study, two type of groove configurations were made i.e. grooves around the beam cross-section (on all sides) and grooves on only three sides, excluding the top face of the beam, as presented in Fig. 1 (b) and (c). The size of the groove was 5 mm wide and 18 mm deep. As illustrated in Fig. 1 (d), the grooves on the north and south faces of
the beam (vertical grooves) were made with 5 mm offset from the horizontal grooves in order to provide an overlapping for each adjacent groove at the beam’s corners, which allowed the laminate strips to interfere with each other by a distance equal to the width of the laminate to increase the strengthening at the corners. The dimensions of the laminate strips were 9.6 mm wide x 1.4mm thick in order to provide the same FRP percentage as the EBR FRP sheet strips 50 mm wide x 0.176mm thick used for the same beam dimensions by Hii and Al-Mahaidi 2004 [13] in order to compare the percentage improvement in torsion with their experimental results.

Eight electrical resistance strain gauges were used to monitor the strain for each beam and verify whether yielding in the steel stirrups had occurred. The 5 mm in length strain gauges placed on each side of the stirrups (north and south legs), and titled S1-S4 and S5-S8 respectively as shown in Fig.2. Chasing the grooves into the concrete cover was initiated after 28 curing days for the beams. Dust and any loose materials were first removed using an industrial vacuum cleaner and brush to clean the grooves. A steel blade was used to fill the grooves with cement mortar after wetting the surface with water, and thin layers of the mortar were applied to the faces of the CFRP laminate strips after removing the peel ply to ensure that the surfaces of the laminates were fully covered with mortar. The laminate strips were then introduced to the required depth and the excess mortar was removed. The beams with the mortar grooves were cured using moistened geotextile cloth for 21 days, and little change in the properties could be observed after 21 days (Al-Bayati and Al-Mahaidi 2014 [2]). The beams were labelled as F, U1, U2, C1 and C2 for the four sides strengthened beam, the first and the second U strengthened beams and the first and the second control beams respectively.
2.2. Material properties

The concrete used for all five beams was mixed and delivered by a local supplier. Table 1 presents the mechanical properties for all materials. The modified cement-based adhesive with the mix proportions and preparation method stated in Al-Abdwais 2015 [12] was used as an alternative to epoxy. The mortar comprised ordinary Portland cement, micro cement, silica fume, silica filler 200G, MBrace Primer and super-plasticizer (Visocrete 5-500). The required viscosity and the workability of the mortar are provided by the primer and super-plasticizer ratios. The elastic modulus (Es), yield and ultimate strength (fy, fu) of the steel bars were determined by testing three bars with 600mm length of each diameter using uniaxial tensile tests were carried out according to ASTM A370-10 [14]. MBrace laminate provided by BASF Chemical Company Ltd. with a rough surface was selected to strengthen the beams. The compressive strength, the tensile strength, and the modulus of elasticity of the concrete and mortar were determined on the first day of the test, and the average values are presented in the table. AS 1012.9 (Standards Australia, 1999 [15]) and AS 1012.10 (Standards Australia, 2000 [16]) were adopted for the compressive strength and splitting tensile tests, respectively. Five samples of CFRP laminate strips were tested for tension where Table 1 also presents the average test results of the CFRP strips.

<table>
<thead>
<tr>
<th>Material</th>
<th>Age (days)</th>
<th>Compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Modulus of Elasticity MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>245</td>
<td>49</td>
<td>3.534</td>
<td>38520</td>
</tr>
<tr>
<td>Mortar</td>
<td>91</td>
<td>69</td>
<td>5.2</td>
<td>30320</td>
</tr>
<tr>
<td>Laminate adhesive</td>
<td>-</td>
<td>-</td>
<td>21.3</td>
<td>10000</td>
</tr>
<tr>
<td>Steel bars Ø12</td>
<td>-</td>
<td>-</td>
<td>Fy: 540.5 Fu: 673.7</td>
<td>207700</td>
</tr>
<tr>
<td>Steel bars Ø6</td>
<td>-</td>
<td>-</td>
<td>Fy: 352 Fu: 460</td>
<td>232000</td>
</tr>
<tr>
<td>FRP laminate</td>
<td>-</td>
<td>-</td>
<td>Fu: 3600</td>
<td>212400</td>
</tr>
</tbody>
</table>

Table 1. Mechanical properties of the used materials.

2.3 Test set-up details

Fig.3 (a) and (b) present the torsion rig system that was used to test the beams. A steel frame with an external clamping collar was utilized to provide a fixed support for the beams at the east end, while the torque was applied through a hydraulic actuator and a steel loading arm at the west end of the beams. In order for the beam to twist freely while releasing the elongation or shortening, axial movement was permitted by using a roller seated on a linear bearing with a spherical ball head to support the beam where the load was applied. Another spherical ball was located over the cantilever arm and under the hydraulic actuator to ensure that the load remained vertical during the test. The alignment of the roller seat with the loading arm and its spherical ball were checked to produce pure torsion and to ensure that no other bending or shear forces were introduced into the system. The angle of twist was measured using inclinometers located at the East and West ends of
the beams. A displacement control loading with a load rate of 1mm/minute was adopted for the testing with a frequency of 3 Hz for recording. Eight electrical resistance strain gauges were used to monitor the strain for each beam and verify whether yielding in the steel stirrups had occurred. Fig. 3 shows the four strain gauges 5 mm in length placed on each side of the stirrups (north and south legs), and titled S1-S4 and S5-S8 respectively.

3. Results and discussion

3.1. Torque-twist curves and failure mode
The resulting torque versus angle of twist curves for all five beams are presented in Fig. 4. For all beams, linear elastic behavior was initially detected followed by a considerable increase in the angle of twist with a gradual increase in torque until failure. After ultimate, the curves show a gradual decrease in the torque until the end of the test while the angle of twist continued to increase. The crack and ultimate torques were extracted from the curves for each beam, where the ultimate torque is the maximum torque beyond which the beam will fail and the crack torque is the torque when the first diagonal crack appeared. For the reference beams C1 and C2, the first crack appeared at the top then the north face and later progressed to the other two faces. The crack torques were 4.72 kN.m and 4.30 kN.m for C1 and C2 respectively. The crack strengths of the two beams were notably different, while the ultimate strengths were similar. Comparable torsional behaviors were observed for all the strengthened beams. Higher crack torques were achieved for the strengthened beams. It was observed that the first cracks developed at the north face then followed at the other three faces for beam F. For beams U1 and U2, the first and the critical cracks initiated at the unstrengthened face of the beams. Fewer cracks were found at the other three faces of the beams compared with beam F. For all beams, the post-cracking stiffness gradually softened with increased torque up to failure. The post-cracking stiffnesses of all strengthened beams were higher the post-cracking stiffnesses of the control beams, which can be attributed to the contribution of the FRP in carrying the additional torque. Beam F had the largest torsional capacity of 7.83 kN.m.
The crack torques, ultimate torques and corresponding angles of twist of the five beams are shown in Table 2. The average values of the torques and twists for the corresponding beams were calculated and the results are illustrated in the table, and the increased percentages over the control beam are also provided between brackets. The average crack torques of beams U1 and U2 were higher than the reference beams by about 15%. Beam F recorded a higher improvement in the crack torque with a highest corresponding angle of twist. It was stated in previous research that the crack torque and the angle of twist increased as a result of using EBR CFRP strips (Hii and Al-Mahaidi 2004 [13]). Although the experimental work showed an improvement in the crack torques of the strengthened beams, no specific tendency can describe the crack torque behavior of the strengthened beams with respect to the control beams using the NSM method, unlike the EBR technique. This is because the mechanism of making grooves around the beams’ cross-section can cause a slight reduction in the cracking stiffness of the beams, and, on the other hand, a stiffer pre-cracking behavior could be also expected as being attributed to the FRP strips. The average ultimate torque of the reference beams was 6.77 kN.m at an average angle of twist of 3.76 degrees. The average ultimate torque of beams U1 and U2 was increased by 12.7% at an angle of twist of 3.918 degrees. Beam F had the largest ultimate strength of 7.83 kN.m at a twist angle of 3.358 degrees. The increased percentage was 25.2% for the beam strengthened with the CFRP EBR strips spaced at 200 mm (Hii and Al-Mahaidi 2004 (10)), while the use of CFRP NSM laminates with the same FRP percentage provided a 15.7% increase in the ultimate torsional strength of the beams. Using a U-configuration groove in beams U1 and U2 decreased the increased percentage in the ultimate strength of about 3% for beams U1 and U2 compare with beam F (12.7% versus 15.7%). Although the use of mortar as an alternative to epoxy provided a less increased percentage of ultimate strength, the benefit of its use to avoid problems such as hazardous toxic fumes and the deterioration of strength with higher temperatures might be considered more significant.

Photos taken after the test depicting the failure zones of beams U1, U2 and F are shown in Fig.5. No specific trend was observed for the exact location of failure, and the location of first crack initiation and failure could be related to factors such as reinforcement spacing, non-homogeneity of concrete properties and other factors relating to imperfections in beam fabrication. The failure mode of the control beam was by yielding of the steel closed ties followed by local concrete cover spalling and concrete crushing. When the
maximum torque was attained, the torsion failure crack suddenly widened. A few large cracks were observed and the largest cracks were located in the vicinity of the concrete spalling zone. For beam F, the failure mode was by yielding of the steel closed ties followed by local concrete cover spalling. Diagonal cracks were observed where the laminate crossing the path of the torsional crack failed by debonding. Interface cracks between the concrete and the mortar and small uniformly distributed cracks were first observed and they then developed into the groove corners, ending with separation of the concrete cover including the CFRP strips. The larger cracks developed in the area of the beam between the second and third grooves at the west side of the beam (the load side) passed through the grooves, particularly the grooves’ corners, as they are the weakest point for dis-continuous laminate. For the beams using the U-strengthening technique, the failure mode was by yielding of the steel closed ties followed by concrete crushing at the unstrengthened face. Similar initial trends were formed compared to beam F. However, fewer cracks were found in each beam. Small uniformly distributed cracks occurred at the three strengthened faces in the space between grooves. Interface cracks were also observed between the concrete and the mortar.

![Figure 4. Torque-twist curves.](image)

<table>
<thead>
<tr>
<th>Beams</th>
<th>Crack torque (kN.m)</th>
<th>Average</th>
<th>Twist at crack torque (°)</th>
<th>Average</th>
<th>Ultimate torque (kN.m)</th>
<th>Average</th>
<th>Twist at ultimate torque (°)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>4.72</td>
<td>4.51</td>
<td>0.168</td>
<td>0.183</td>
<td>6.77</td>
<td>6.77</td>
<td>4.26</td>
<td>3.76</td>
</tr>
<tr>
<td>C2</td>
<td>4.30</td>
<td></td>
<td>0.198</td>
<td></td>
<td>6.77</td>
<td></td>
<td>3.27</td>
<td></td>
</tr>
<tr>
<td>U1</td>
<td>5.60 (+15.%)</td>
<td>5.19</td>
<td>0.273</td>
<td>0.240 (+31.1%)</td>
<td>7.63</td>
<td>7.62 (+12.7%)</td>
<td>3.645 (+4.1%)</td>
<td></td>
</tr>
<tr>
<td>U2</td>
<td>4.79</td>
<td>0.207</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.191</td>
<td></td>
</tr>
</tbody>
</table>

Table 2. Crack and ultimate torques.

### 3.2 Strains in the steel stirrups

The results for the specific strain gauges on the steel stirrups in beams U1 (S1) and F (S4-S8) were cancelled as they were damaged during manufacture of the beams or the groove chasing. The yield strain for the stirrups was 0.001517. In all beams, the strains exceeding the yield strain were located at the stirrups.
intersected by large torsional cracks. For example, the strain readings of gauges S1 and S2 in the control beam C2 and gauge S6-S8 in beam U1 were relatively small, as no large cracks intersecting the stirrups were observed. In Fig.6, examples of the torque-strain curves for all strain gauges are shown for beams C2, U2 and F. The figure clearly shows that the individual reinforcement strains exceeded the yield strain before reaching the ultimate load for each beam. For example, beam U2 has eight strain gauges (S1, S2, S3, S4, S5, S6, S7 and S8) exceeding the yield at different load steps under the ultimate torque of 7.62 kN.m, whereas gauge S2 reaches the yield strain first at a torque of 6.22 kN.m, while gauges S3, S4 and S5 are the last gauges before ultimate torque to reach the yield strain at torques of 7.05 kN.m, 6.66 kN.m and 6.98 kN.m respectively. At low levels of torque, the stirrups experienced low strains, as is evidenced by the readings. After spiral cracks started to form in the beams, the reinforcement strain values increased rapidly. The stiffness of the beams was reduced with increasing crack opening, as more force was gradually transferred to the stirrups. Under the same loading conditions, the strain levels measured in all the strengthened beams were generally lower than in the control beams. In addition, the application of FRP laminate strips on all beam sides (beam F) resulted in smaller reinforcement strains compared to beams U1 and U2. This is an indication that the CFRP laminates on the top face of the beam carry an extra component of torque. Table 3 shows the strain values of stirrups measured at ultimate torque. The strain of the stirrups was attributed to the fact that only one side of some steel stirrups had been instrumented with strain gauges, and to the likelihood of the peak reinforcement strain occurring in the unmonitored regions of the bars.

![Images of beams U1, U2, and MF](images)

**Figure 5.** Failure zones for beams U1, U2 and F.
Figure 6. Strain versus angle curves for beams.

<table>
<thead>
<tr>
<th>Strain gauge</th>
<th>Strain × 10⁻⁶</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>S2</td>
</tr>
<tr>
<td>C1</td>
<td>715</td>
</tr>
<tr>
<td>C2</td>
<td>76</td>
</tr>
<tr>
<td>U1</td>
<td>-</td>
</tr>
<tr>
<td>U2</td>
<td>942</td>
</tr>
<tr>
<td>F</td>
<td>1771</td>
</tr>
</tbody>
</table>
4. Numerical predictions

In order to shed further light and to develop advanced theoretical understanding of NSM torsional strengthening system with mortar, the primary analysis of reinforced concrete beams using non-linear finite element model is presented herein as a good perception into the behavior of the beams can be provided using an advance FE analysis. The early model is carried out through the use of ATENA 3D software (Cervenka and Cervenka 2010 [17]). 3D Nonlinear Cementitous is a fracture plastic model in ATENA 3D used for brittle material such as concrete and mortar. Automatic mesh generation is used where each part of the structure was meshed separately. The creation of the performed model in ATENA 3D included the definition of material models for concrete, steel reinforcement, FRP laminates, the adhesive (mortar) and an interface bond law between the adhesive and the concrete. Torque-twist curves were examined in this primary analysis to determine if the predicted behavior was in good agreement with the experimental observation. The torque-twist curves for control and strengthened beams obtained by nonlinear finite element analysis (NLFE) are illustrated in Fig.7. The torque-twist curves were plotted up to failure where no changes to the step sizes or convergence criteria were considered. The crack strength and corresponding angle of twist for the control beams were predicted higher than the experimental results by 17% while the ultimate strength was found to be more or less identical to the experimental result with lower angle of twist by 16%. For the strengthened beam models with U configuration, the ultimate strength was predicted with a high precision with lower corresponding angle of twist by 9%. The accuracy of predicting the crack strength was within -2%, nevertheless, the margin of difference of the angles of twist values were -10%. The ultimate strengths for the beams strengthened at all faces were predicted with high accuracy while the predicted corresponding angles of twist were lower than experimental values by 9%.

Yield strain of stirrups = $1517 \times 10^{-6}$

Table 3. Strain values in the stirrups at peak torque.
Conclusions

The experimental results showed the influence of using different strengthening configurations of the NSM technique with cement based adhesive for torsional strengthening of reinforced concrete beams. The results of strengthened beams were evaluated and compared with those of the reference beams. The ultimate capacity of the beams for torsion improved by almost 12.7% and 15.7% for the four-face and three-face strengthening configurations of CFRP NSM laminate strips respectively. The crack torques of the strengthened beams were generally higher than those of the reference beams. However, no specific tendency can be attributed for the crack capacity of the beams, as making grooves and adding FRP strips have two opposite effects on initiation of the first crack. It is clear that using FRP strips on the all beam faces provides higher torsional strength. However, an acceptable margin of improvement compared to the control beams was achieved using the U-shaped strengthening system. Although the torsional improvement was lower, the use of the U-shaped strengthening configuration may be more applicable to many cases and the benefit of using mortar as alternative material to epoxy might be considered more important to avoid many problems. At the same loading conditions, it was observed from the stirrup strain gauge readings that the torque carried by the steel bars in the fully face-strengthened beams was generally small compared with the corresponding beams strengthened with U-shaped CFRP laminates or with the reference beams. The critical cracks for strengthened beams with the U-shaped configuration were developed at the un-strengthened face, causing the concrete cover to spall. However, the critical torsional cracks where failure of the beam occurred for the beams strengthened on all faces with interface cracks developed through the corners of the laminate strips, as they are the weakest points, and resulted in concrete cover delamination. Further investigation for the use of the NSM technique with torsional strengthening is required to study other parameters, which could increase the contribution of CFRP to strengthening. Finally, it can be observed that good agreements for the ultimate and crack capacities has been achieved between the experiment and the primary numerical predictions.

Acknowledgement

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References


Experimental Study on the Hydraulic Performance of Trapezoidal Planform Compound Labyrinth Weir

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Labyrinth weir is considered one of the most efficient solutions to regulate upstream elevations of water and hence increasing in flow capacity through increasing the crest length without changing the existing width of the structure. The trapezoidal planform of Labyrinth Weir is common shape used to regulate the water flow. The major aim of the present study is to investigate the hydraulic performance of a newly developed geometry of compound labyrinth weir under free flow conditions in the rectangular laboratory flume. In this study, 10 lab scale models are fabricated to investigate major design parameters which affect the weir efficiency for sidewall angle, weir orientation and weir height. The compound discharge coefficient for flow through holes and over trapezoidal labyrinth weir will be quantified from the test measurements. The curves were found from experimental data that show the relationship between the discharge coefficient and dimensionless term HT/P of one cycle compound labyrinth weir. The result shows that reducing the weir height result in increasing the coefficient of discharge (Cd) values for a given total head over weir height (Ht/P). Test results confirmed that the value of Cdc does not change significantly when comparison orientation of labyrinth weir.

1. Introduction

A labyrinth weir is a barrier built across a channel to increase the water level on the upstream side and it is allowing the water to pass over all crest length to the downstream side. There is a need to improve the capacity of discharge of the existing labyrinth weirs due to increasing demand for storage capacity, increasing events of floods etc. Labyrinth weirs consist of a series of linear weirs which are folded in plan view to give a longer crest length compared with a normal weir that having the same width to increase the flow for a given water head. labyrinth weirs deliver large flood at a comparatively low head. therefore, it can be commonly utilised to pass a range of discharge with a limited variation in upstream water levels and also where the width of a channel is limited. Discharge (Q) over a labyrinth weir under free flow condition can be expressed by the following mathematical expression

\[ Q = Cd \times \frac{2}{3} \sqrt{2g \times L_c \times H_T^{1.5}} \]  

(1)

Where Cd = discharge coefficient, Lc = length of the labyrinth weir crest, Ht = total head over the crest, g = acceleration due to gravity. The Cd depends on the geometry of the channel and the labyrinth weir and flows characteristics [1].
[2,3] examined different shapes of labyrinth weirs and obtained the results in the form set of curves that it represents the relationship between the discharge ratio over labyrinth weir (Q) to corresponding traditional normal weir (Q) and h/p. where p = weir height. The outcomes for their study explained that the triangular labyrinth weir is more efficient than the trapezoidal labyrinth weir. [4] studied two-cycle Hyrum Dam auxiliary labyrinth spillways and investigated labyrinth weir positions and orientation effect on discharge. He found an effect on the head values in the normal position slightly less than those comparatives to the inverted position. [5] demonstrated the labyrinth weir capacity is function depend on the discharge coefficient, total head and length of the effective crest. [6] illustrated dimensionless submerged head parameters to describe the relationship water head and discharge of the submerged labyrinth weir with sidewall angles of labyrinth weir. [7] suggested a methodology for the best value of the coefficient of discharge for design labyrinth weirs. [8] conducted tests on different crest shape of the weir and utilising dimensionless analysis, he proposed the equation for calculating discharge over labyrinth weir. [9,10] investigated the flow characteristics over triangular labyrinth weir and found the efficiency the triangular labyrinth weir is better than the traditional linear weir. [11] carried out experiments on a characteristic of flow over trapezoidal labyrinth weirs by utilising experimental work for a range of sidewall angles of 8° to 30°. [12] investigated the hydraulic performance of labyrinth weirs. They studied effect crest elevation on the coefficient of discharge for labyrinth weir. [13] conducted experimental work for five models of rectangular labyrinth weir with gate. They examined affect the effective length and height of labyrinth weir with various slopes of flume on discharge coefficient. They studied two cases for flow under and over the rectangular labyrinth weir-gate. The following equation was used to calculate the discharge coefficient for compound labyrinth weir that derivated by depending on flow over crest labyrinth weir and the flow through the holes that located it on wall labyrinth weir [14]

\[
Q_{act} = C_{dc} \left[ \frac{4}{3} \sqrt{\frac{2g}{3}} * b_1 \right. * H_{T}^{1.5} + \frac{32}{25} * \sqrt{\frac{2g}{5}} * \tan \frac{\theta}{2} \left. * H_{T}^{1.5} + \frac{2}{3} \sqrt{2g} * L_c * H_{T}^{1.5} \right] \quad (2)
\]

Where \( C_{dc} \) = compound coefficient of discharge, \( L_c \) = length of labyrinth weir crest, \( g \) = acceleration due to gravity, \( H_t \) = total head over the crest.

2. Experimental Setup

2.1. Instrumentation

This work conducted in a civil lab at Deakin University. All the models setup in the tilting rectangular laboratory flume that have dimensions (25 cm depth, 7.5 cm width, and 500 cm length). the flume walls made of acrylic panels and a steel framework. Adjustment bed slop by one jack manually. In this study, the longitudinal slope for flume bed was set to zero. The water supply to flume was through the flexible pipe with diameter 2 in (5 cm) as shown in (Fig. 1). There is a sluice gate in downstream of the flume to regulate and control on the tail water level. The flume contains on a tank with capacity 250 L. The flow capacity is rated between 10 - 150 L/min. The water level is measured using point gauges with an accuracy of 0.1 mm and located (3h) upstream of the weir. All models are setup in the flume at a distance of 1 m from the inlet point. A digital flow meter is used to measure rates water flow. A thermometer is used to measure water temperatures with a range of 58°F to 302°F and readable to ±0.05°F.
2.2. Model Description

A 10 models configurations were examined in the present study. The models were built of wood, thickness 0.5 cm and coating as shown in (Fig. 2). The form of models were trapezoidal compound labyrinth weirs. The details of the physical model are shown in the (Table 1). Each model was examined in reverse and normal flow orientation. [15] stated that normal orientation when the outside apexes of a labyrinth weir fix to the flume wall at the upstream. While it is inverse orientation when apexes fix to the flume wall at the downstream end of the apron.

![Figure 2. Tilting rectangular flume in civil lab at Deakin University](image)

<table>
<thead>
<tr>
<th>Model</th>
<th>α degree</th>
<th>P (cm)</th>
<th>B (cm)</th>
<th>Lc (cm)</th>
<th>Tw (cm)</th>
<th>A (cm)</th>
<th>D (cm)</th>
<th>N</th>
<th>B1 (cm)</th>
<th>B2 (cm)</th>
<th>Y (cm)</th>
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<td>Trap.</td>
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<td>Normal</td>
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</table>

*Table 1. Physical model test program.*
Where: $\alpha$ is sidewall angle, $P$ is height labyrinth weir, $B$ is long labyrinth weir, $L_c$ is crest length, $T_w$ is wall thickness, $A$ is inside apex width, $D$ is out side apex width, $N$ is number of cycles, $B_1$ is bottom width for hole, $B_2$ is top width for hole $Y$ is hole depth.

3. Results and Discussions

In this study investigated the discharge coefficient of compound labyrinth weirs under free flow conditions. The first group of tests were conducted on the one-cycle models of trapezoidal compound labyrinth weir and used flat weir crest and four sidewall angles $16^\circ, 21^\circ, 30^\circ$ and $90^\circ$ leaner weir for comparing. The second group the tests carried out on labyrinth weirs have sidewall angle 20o and 10, 12, 15 cm weir height to examine the effective height of weir on the coefficient of discharge. 10 readings took for each run. The flow was measured for each run between 15 L/min to 150 L/min.

The current study applied the equations (Eq. 2) to compute the compound discharge coefficient. (Fig. 3), (Fig. 4) shows the relationship between the compound discharge coefficient $C_{dc}$ with dimensionless term $HT/P$ for inverse and normal direction for the flat labyrinth weir crest. In these figures seen that there is a maximum value for the compound coefficient of discharge in each of the curves. Then it follows by the long falling limb of the curve. The angles, $\alpha^\circ = 90^\circ, 30^\circ, 21^\circ$ and $16^\circ$ at $HT/P = 0.15$, the compound coefficient of discharge is increased slightly because the nappe flow was the sudden removal of the air cavity behind the nappe [15]. Also, the compound coefficient of discharge values are reduced with increasing $HT/P$ due to the nappe of flow from nearby crests collides with another sidewall due to a non-aerated nappe (e.g., $HT/P = 0.35$ for $\alpha = 16^\circ$), also see (Fig. 5).

![Figure 3. The relationship between $C_{dc}$ values and $HT/P$ for flat crest of trapezoidal compound labyrinth weir](image3)

![Figure 4. The relationship between $C_{dc}$ values and $HT/P$ for flat crest of trapezoidal compound labyrinth weir](image4)
The compound coefficient of discharge is increased when the sidewall angle ($\alpha^\circ$) increases because reducing the crest length for the restrict width of labyrinth weir, hence reducing the effective region of the nappe interference. The results showed that the values of Cdc do not change significantly when comparison orientation labyrinth weir because the data are collected from examines one cycle labyrinth weir. The number of the apex is equal in both upstream and downstream of labyrinth weir that led to the nappe interference is less effective as shown in (Fig. 6).

(Fig. 7) shows comparison values of Cdc versus HT/P for different crest heights (P) for the flat crest of trapezoidal compound labyrinth weirs when (\(\alpha\)) is 20 degrees. The compound discharge coefficient is increasing with reduced crest height of labyrinth weir for the same water head over the crest and reduces with the rise of water head over the crest of the labyrinth weir because of the interference of the water jets in downstream [16].

![Figure 5](image1.png)

**Figure 5.** Explain nappe interference. (a) before the submerge for $\alpha^\circ = 21^\circ$. (b) nearby crest that nappe observe collisions with nappe of other side for $\alpha^\circ = 16^\circ$.

![Figure 6](image2.png)

**Figure 6.** Comparison between the normal and inverse orientation of labyrinth weir.

![Figure 7](image3.png)

**Figure 7.** Comparison values of Cdc versus HT/P for various (P) for flat crest trapezoidal labyrinth weir and $\alpha = 20^\circ$. 
4. CONCLUSION

Labyrinth weirs are a hydraulic structure common in the world because of favorable in increasing ability the weir to pass greater amounts water through it in the flood seasons. The labyrinth weir is reflected an effective method for the improving in the storage capacity. From above results , The conclusions are obtained as the following:

1. The values of discharge coefficient were obtained from the design curves depend on tests that conducted on labyrinth weirs with four sidewall angle 16,21,30,90 degrees and flat labyrinth weir crest.
2. The compound discharge coefficient is increased initially when the water level over crest labyrinth weir reaches a higher value, then the curve gradually climbs down.
3. The results shown that the compound coefficient of discharge is the low value for a sidewall angle of 16° and increases with high value for the sidewall angle under the limited width of the labyrinth weir.
4. The effect of orientation labyrinth weir was assessed. The present investigation demonstrates that labyrinth weir orientation does not significant effect on the discharge efficiency because the experimental data is obtained from one cycle labyrinth weir.
5. The compound discharge coefficient decreases with increasing crest height for the given water head over the crest.

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Reference


Investigation of the Relationship between Ambient and Actual Temperatures in Concrete Bridge Decks

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Keywords: Expansion Joints, Bridge Movements, Bridge Temperature, Thermal Movements, Thermal Expansion, Concrete Bridges.

Expansion joints are essential to account for movements in bridges such as those produced due to thermal variations. Current design methods used to select suitable joints are standards which have been proven to overestimate these movements which results in increased costs. This study was carried out to estimate thermal movements correctly by predicting the relationship between bridge temperature and ambient temperature accurately. Current procedures developed by various researchers used to explore this relationship were investigated and their limitations identified. This was followed by investigating the thermal behaviour of bridge cross-sections. A total of 14 concrete cross-sections have been modelled by finite elements using the software Strand7. The results were compared to field data and to temperature data. The data utilised for this study consisted of thermal movements measured from a bridge in Melbourne and temperature data obtained from the Australian Bureau of Metrology. The comparison of outcomes of the analysis with field measurements and with ambient temperature resulted in finding a more accurate relationship between bridge movements and the ambient temperature. Finally, movements calculated using the developed relationship showed good agreement with actual movements and that indicated the accuracy of this procedure compared to methods of previous researchers.

1. Background Information

On a typical day, the outdoor temperature varies from a low temperature in the early morning to a higher temperature in the afternoon and then back to a lower temperature in the evening and early morning. This commonly identified diurnal temperature variation is incorporated in the design of bridge structures, since it produces thermal movements that must be accounted for. These movements are catered for by expansion joints and bearing, and if these are determined inaccurately, unsuitable joints may be selected and this can increase costs [1]. Thermal movements are produced from variations in bridge temperature which occur due to fluctuations in the ambient temperature. According to Emerson cited in [1] the relationship between actual and ambient temperature in bridges is not linear, as the bridge cross-section will not heat up or cool down at the same rate and time as the ambient temperature. Instead, the temperature has a complex distribution inside a bridge’s cross-section and this complicates the determination of temperature-induced movements [2]. Hence, the question that comprises the basis of this research is “How does the temperature of a bridge vary with the ambient temperature?” Therefore, the aim of the project is to determine the relationship between ambient temperature and bridge temperature to enable accurate calculation of thermal movements.
2. Literature Review

2.1. The Equation for Longitudinal Thermal Movements

The current method used for designing for thermal movements in bridges is the equation specified by AS5100 [3] for longitudinal thermal movements, which is as follows:

\[ \Delta = \alpha \cdot L \cdot \Delta T \]  

(1)

Where \( \Delta \) is the thermal movements in mm, \( \alpha \) is the coefficient of thermal expansion, \( \Delta T \) is the relative temperature of the bridge in °C and \( L \) is the length of the bridge in mm. In Eq. 1 above, the length of the bridge \( L \) and the coefficient of thermal expansion \( \alpha \) are constants, meaning that the only variable in the equation is the temperature of the bridge \( \Delta T \) which is affected by the ambient temperature [3]. However, bridge temperature varies not only with the ambient temperature, but also across the depth of the cross-section of the bridge. As mentioned earlier, since the temperature distribution inside a bridge’s cross-section is complex, Emerson cited in [1] proposes that the section can be divided into three parts according to that distribution. Emerson also assumes that the temperature of the median portion of these three hypothetical parts depends on the weather of the previous two days from which a measurement is taken. This means that a time factor (time lag) influences the relationship between ambient and bridge temperature [4]. As a result, Eq. 1 is not sufficient to characterise the time- and depth-dependent relationship between ambient and bridge temperature, since a single value for \( \Delta T \) is specified against a range of values for the ambient temperature [2].

2.2. Methods to Predicate the Relationship Between Ambient and Bridge Temperature

Several studies have been presented on the relationship between the ambient and bridge temperature, in order to incorporate it in the design for thermal movements. Earlier approaches to finding that relationship included the establishment of contour maps based on maximum and minimum temperature data obtained for a substantial amount of time in a certain location [1]. These design maps were a result of methods proposed by Kuppa and Emerson cited in [1], which were a simplification of previous heat flow analysis procedures. Nonetheless, these maps depended upon extreme weather conditions only which made them invalid, since a bridge’s temperature can never reach the extreme ambient temperature value due to thermal inertia [5]. Other researchers presented a theoretical or an empirical method to find a relationship and then they compared their results with a set of field measurements. The field measurements are either for the actual movements or for the temperature of the bridge, given that these measurements would be for certain bridge in a certain location of a certain climate. Froli et al. and Barr et al. [6, 7] obtained field data from the thermal monitoring of a prestressed concrete box girder bridge. Froli et al. used thermistors at the inner and outer surfaces of different segments across the bridge and found that the longitudinal heat flow can be neglected for bridges with constant or moderately variable cross-sections. Rodriguez [8] also used a similar technique to measure the temperatures of two other bridges and the results showed that the temperature of a bridge varies across its cross-section as a function of time. Thus, the temperature varies across the cross-section of a concrete bridge as a function of depth and time. Alternatively, Parkinson, Taplin and Connal [5] simply measured the temperature movements of the Gateway Bridge by installing string potentiometers (string pots) under the expansion joints of the bridge. Nonetheless, the field measurements, either bridge movements or temperatures, are mainly used to validate the results of the methods proposed by the aforementioned researchers. The most recommended method is to perform a two-dimensional transient heat analysis to model and analyse heat transfer between the ambient and the bridge and compare its results with field measurements, in order to confirm the validity of the analysis.
[2, 6, 9, 10]. This approach relies upon the boundary conditions inserted into the model, which simulate heat exchange between the ambient and the bridge. Since the heat transfer process occurs mostly due to convection, the most important boundary condition is the convective coefficient, as it governs that process. However, none of the researchers directly highlight the importance of this coefficient. Moreover, no consistent equation is provided to predict the relationship between ambient and bridge temperature. One group of researchers [6] conducted two dimensional transient heat finite element (FE) analysis based on the Casilina Bridge (concrete box girder). The boundary conditions incorporated in their model were the ambient temperature, wind speed and solar radiation, which was identified as an important component in the heat transfer process. The ambient temperature measurements (acquired from field data and from a weather station) included in the model were taken from only three periods (August 1987, July 1989 and November 1987), and each period consisted of seven days of steady air temperature distribution and clear sky conditions. For each period, the diurnal variation of the ambient temperature was calculated by considering the heat transferred only by convection. When the results of the FE analysis were compared with bridge temperatures obtained from field monitoring, it was observed that the difference between them did not exceed 3°C [6].

![Figure 1. Comparison between measured (M) and calculated (T) temperature [6].](image)

This can be seen in Fig. 1, where the dotted lines represent the calculated temperature and the other lines represent the measured temperature. As a result, it is assumed that the transient heat FE analysis with the boundary conditions proposed by the researchers is sufficient for the prediction of the temperature variation of concrete bridges. However, as previously stated, a consistent and reliable equation for determining bridge temperatures was not provided.

3. Methodology

To find a relationship between the temperature of a concrete bridge and the ambient temperature, which is the focus of our research, a transient heat FE analysis of concrete cross-sections was carried out using the software Strand7. Field data of bridge thermal movements were also recorded to validate the results.

3.1. Field Data

The field measurements that were utilised were obtained from a bridge in Melbourne. The measurements comprised of bridge movements that were recorded by installing string pots under the four expansion joints of the bridge. Each pair of string pots was connected to a data acquisition unit (Data Taker DT-80), in order to record the data obtained by the string pots over a time interval of 15 minutes. In addition, shade air temperatures inside the bridge were recorded using sensors connected to the data logger. The field measurements were collected from February 6th until April 2nd 2015. Ambient temperature data near the bridge were also purchased from the Australian Bureau of Meteorology (BoM) for the same
period and for the same time intervals of 15 minutes. Hence, adequate data were provided to explore the relationship between ambient and bridge temperature.

3.2. FE Analysis
Before initiating the analysis, the cross-section of the bridge from which the field data were obtained was modelled and analysed. The same boundary conditions applied to all of the other models, which are explained in the following sections, were applied to this model. The results (thermal movements) obtained from the model (model 7-hollow) were then compared to the measured movements obtained from the field and the comparison is shown in Fig. 2.

![Figure 2. Comparison between actual and theoretical displacements of the bridge.](image)

As the figure shows, the maximum difference between the actual and the theoretical data does not exceed 12 mm, which provides an accuracy of 85%. This was done in order to certify the validity and reliability of the models created, the conditions applied to them and the results obtained. This procedure also guarantees the consistency of the results with the measurements and provides assurance that the conclusions and the relationships drawn from them are correct. Fourteen concrete cross-sections were then modelled and analysed using Strand7, half being solid and the other half being hollow cross-sections.

A value for the convection heat coefficient, which governs the heat transfer process, of \( 35 \times 10^{-6} \text{ W/m}^2\text{C} \) was applied to the models. This was found by using the following equation:

\[
\text{Transient Heat Coefficient} = 13.5 + 3.88v \quad (2)
\]

Where, \( v \) is the average wind velocity in m/s [6]. This was found to be 20.1 Km/h (5.58 m/s), which is the average wind speed recorded in Melbourne in 2015 until April (the same period for which the field data were collected). This equation was used because it provided reasonable and consistent results from the analysis performed.

3.3. The Models
Before creating the models, a theory or an idea which could link the models and their characteristics with the temperature of the bridge and ambient air was required. AS5100 part 5 [3] states that for shrinkage control there is a variable known as the hypothetical thickness \( t_h \) which is expressed by:

\[
t_h = \frac{2A_g}{u_e} \quad (3)
\]

Where, \( u_e \) is equal to the exposed perimeter of the cross-section of a member plus half the perimeter of any closed holes that the cross-section contains, and \( A_g \) is the cross-sectional area. This parameter indicates that concrete cross-sections with larger perimeters experience more shrinkage (as it is easier to expel water) for the same cross-sectional area as a concrete section with a smaller perimeter. Hence, the same principle can be applied for temperature, since if two cross-sections of the same cross-sectional area are considered and one of them has a larger perimeter, then heat will transfer faster into that one. To test
the theory, two cross-sections were considered: one was 102x103mm and the other 500x200mm and both had the same cross-sectional area of 105mm. For the first cross-section \( t_h \) is equal to 45.45mm and 71.43mm for the second one. Therefore, \( t_h \) for the first section is smaller and this means that heat transfer occurs more quickly for this section, thus, to explore this idea, two preliminary Strand7 models were constructed. The results of the two models showed that the section with the larger perimeter of 102x103mm (smaller \( t_h \)) was more influenced by the ambient temperature than that of the smaller perimeter of 500x200mm (larger \( t_h \)) and this is shown in Fig. 3 This is because, time lag for the 102x103mm section is much smaller than that for the 500x200mm section.

![Figure 3. Ambient and model (1000x100mm) and (500x200mm) temperature.](image)

To apply the theory proposed earlier “if two cross-sections of the same cross-sectional area and one of them has a larger perimeter, then heat will transfer faster into that one”, 14 models were created, half of them were solid and the other half were hollow. They comprise of six sets, each set have two models of the same area but of different perimeters and the final two models have the dimensions of the bridge from which the field data were collected. To clarify this, Table 1 below shows the dimensions of the solid models.

<table>
<thead>
<tr>
<th>Solid Model</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
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<tbody>
<tr>
<td>Perimeter</td>
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<td>9500</td>
<td>21000</td>
<td>26600</td>
<td>2200</td>
<td>3400</td>
<td>22000</td>
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<tr>
<td>Width</td>
<td>2000</td>
<td>4000</td>
<td>6000</td>
<td>10800</td>
<td>600</td>
<td>1500</td>
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<td>Depth</td>
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<td>750</td>
<td>4500</td>
<td>2500</td>
<td>500</td>
<td>200</td>
<td>3000</td>
</tr>
<tr>
<td>Area</td>
<td>( 3 \times 10^6 )</td>
<td>( 3 \times 10^6 )</td>
<td>( 2.7 \times 10^7 )</td>
<td>( 2.7 \times 10^7 )</td>
<td>( 3 \times 10^7 )</td>
<td>( 3 \times 10^8 )</td>
<td>( 2.4 \times 10^7 )</td>
</tr>
</tbody>
</table>

**Table 1. The dimensions of the solid models in mm.**

3. Results and Discussion

Results of the analysis were extracted as temperature data that varied across the depth of the model and with time. The temperature data were analysed and examined and then compared against the ambient temperature from the BoM. The dissimilarity was obvious since there was a time lag and a difference in amplitude between the two. This was done in order to modify the ambient temperature to be as similar as possible to that of the models. The modifications included taking a time average of the values of the ambient temperature and multiplying it by a unit less factor (0.1–1). Hence, the factor and the time average transforms the ambient temperature into temperature of a concrete cross-section of certain dimensions. The factor and time average were selected for all of the analysed models by using the trial and error method. The relationships obtained from comparing the ambient temperature to the temperature of the models showed that Time average is inversely proportional to the perimeter/area of the cross-section and directly proportional to the depth of the cross-section. While the factor is directly proportional to the perimeter/area and inversely proportional to the depth of the cross-section. From these relationships, the following equations for the time average and the factor have been obtained:
As mentioned previously, AS5100 [3] provides an equation for calculating thermal movements, where the only variable in this equation is the relative bridge temperature which is obtained from a table that provides a value for bridge temperatures based on a range of values for the ambient temperature. This approach over-estimates thermal movements and this is shown in Fig. 4 which demonstrates that the actual thermal movements are much smaller than the movements calculated based on AS5100 [3]. On the other hand, Fig. 4 also displays the modified bridge movements obtained by multiplying the calculated movements according to AS5100 by the ambient temperature averaged over a period of six hours (time average) and by a factor of 0.3 to provide similar results to the measured bridge movements. Therefore, to obtain realistic results, AS5100’s equation for thermal movements needs to be multiplied by a factor and ambient temperature values that are averaged over a certain time (time average).

The modified equation is presented below:

\[ \Delta` = \alpha \cdot L \cdot \Delta T` \cdot f \]  

Where, \( \Delta` \) is the modified thermal movements, \( \Delta T` \) is the ambient temperature averaged over a certain time (time average), \( f \) is the factor and \( \alpha \) and \( L \) are constants. Hence, to provide accurate thermal movements for concrete bridges AS5100’s modified equation can be utilised along with the equations for the factor and time average.

4. Conclusions

Thermal movements are catered for by expansion joints which can increase costs if these were selected based on inaccurate calculations. Thermal movements occur due to variations in bridge temperature which results from diurnal fluctuations in the ambient temperature. Ambient and actual temperature in bridges are not linearly proportional, instead, their relationship is a function of time and depth of the bridge’s cross-section. It was found that the ambient temperature can be related to the bridge temperature by a factor and a time average which are functions of the geometry of the bridge’s cross-section. As a result, the temperature of a concrete bridge can be calculated from the ambient temperature by multiplying it by a factor and a time average. This can then be inserted into AS5100’s equation for calculating thermal movements to provide accurate results.

\[ \Delta` = \alpha \cdot L \cdot \Delta T` \cdot f \]

Earlier (6)
References


Investigation of Energy Dissipation in Gabion Stepped Weirs

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Keywords: Gabion weir, Energy dissipation, Stepped weir, Inclined steps, End sill steps.

Weirs are widely used in hydraulic structures to control and regulate flow in open channels. Gabion weirs are frequently used in rivers, small dams, energy dissipaters, flood control works, and check dams, among other uses, and can be used to build a stepped weir. Dissipation of the energy (kinetic energy) on spillways or weirs is important to protect both hydraulic structures and downstream channels. This study aims to investigate the energy dissipation in gabion stepped weirs. For this purpose, six lab scale models of gabion stepped weirs each with six levels of impervious stepped weirs were investigated, which includes two downstream slopes (1:1 & 1:2, V:H) and three step face geometry (normal, end sill, and inclined steps). The rock fill material is crushed stone of nominal size (13.2 mm–19.0 mm), with a porosity of 0.42. Results showed that gabion weirs have more energy dissipation than impervious weirs around 20% at low discharge and only 3% at high discharge. Moreover, the step face geometry is more effective at medium discharge (nappe and transition flow regimes), while there is no effect at low and high discharge (through and skimming flow regimes). Finally, it was found that the energy dissipation increased with decreases in the downstream slope for all step configurations.

1. Introduction

Weirs are hydraulic structures widely used in water works such as energy dissipation, water regulation and control, flood mitigation, and other purposes. Weirs differ according to the material from which the body of the weir is made. Some have a solid body constructed from an impervious material such as concrete, while other types have a pervious body constructed from non-cohesive agglomerations of natural or broken particles such as rock-fill. A weir type with a porous body constructed from pervious material is a gabion weir, whose structures have stability and flexibility. They are relatively easy to build, using a porous medium, such as rock-fill, as filling material in a mesh grid Salmasi, Chamani [1]. Generally, gabion weir structures are considered to be an economical alternative compared with other types, as the filling material used in body formation is the most abundant and economical material in hydraulic engineering practice.

Several studies were conducted to investigate the flow characteristics and energy dissipation on gabion weirs. Stephenson [2] estimated the energy dissipation on a rectangular and stepped gabion weirs. Different downstream slopes at V:H ratios of 1:1, 1:2, 1:3, and 2:3 with different numbers of steps (two, three, and four steps) were used. The results show that the energy dissipation increasing as the downstream weir slope decreased. Furthermore, the energy dissipation was increased as the number of steps increased for the first three steps, then decreased with increasing steps. Peyras, Royet [3] conducted an experimental study to investigate the energy dissipation over stepped gabion weirs. Three downstream slopes of 1:1, 1:2, and 1:3, with different numbers of steps (3, 4, and 5). Four step face geometry were used: plain gabions, plain gabions with a horizontal concrete, inclined gabions with a concrete slab, and rectangular gabion end sill steps. Results indicate that inclined and end sill steps had more energy...
dissipation than normal steps. Kells [4] point out that there was no clear effect for downstream slopes on the rate of energy dissipation. In addition, energy dissipation increased up to 20% when through-flow was allowed. Chinnarasri, Donjadee [5] showed that the gabion stepped weirs had more energy dissipation than horizontal stepped weirs, of 7%, 10%, and 14% for slopes 30°, 45°, and 60°, respectively. Moreover, the downstream slope had more effect on energy dissipation than the stone size and shape. Salmasi, Chamani [1] investigate the behaviour of gabion stepped weirs for energy dissipation. Results show that at low rate of flow, the dissipation of energy increased as increasing rock-fill porosity. Moreover, decreasing the downstream slope of the spillway produced more energy dissipation. Wuthrich and Chanson [6] studied the hydraulic performance of gabion stepped weirs. They demonstrated that at the high rates of discharges, the gabion stepped weirs had less energy dissipation than the impervious weirs, and this result may be a counterintuitive result Wuthrich and Chanson [6], Chanson [7]. In contrast, at low discharge, the stepped gabion weirs had a higher energy dissipation than the impervious weirs. In summary, several studies on gabion stepped weirs were conducted, but the results showed some differences, especially in terms of the effect of downstream slope on the energy dissipation, in addition to the lack of sufficient studies on the step face geometry and how to improve energy dissipation. The aim of present study is to investigate the energy dissipation on gabion and impervious stepped weir with different downstream slopes and step face geometry.

2. Experimental setup

The hydraulic flume of plexiglass side-wall has 500 cm length, 7.5 cm width, and 25 cm height as were used at the civil laboratory in school of engineering / Deakin university as shown in Figure (1). The water tank capacity was 250 litres with a pumping system of a maximum flow rate of 150 l/min. To control tail water depth and the hydraulic jump position, the flume was equipped with sluice gate at the downstream end. To measure flow rate, a digital flow meter was used with an accuracy of ± 3%. Pointer gauges with an accuracy of ± 0.1 mm were used to measure water depth at three positions: upstream of the weir, and downstream of the weir before and after the hydraulic jump.

![Figure 1. Hydraulic flume details](image)

Three steps, two downstream slopes (1:1, and 1:2) with three steps geometry (normal, inclined, and end sill) were used for gabion and impervious stepped weir. Crushed stone of nominal size (9.5 mm–19 mm) with porosity of 0.42 was used as a rock fill material. All models were designed to a scale of 1:20 and had the same height, width, step height, and broad crest (height 15 cm, width 7.5 cm, step height 5 cm, and broad crest 10 cm). The sketch of physical models was shown in Figure (2).
3. Energy dissipation:
The energy dissipation on the weir can be defined as the difference between the energy of water upstream and downstream of the weir. Figure (3) shows the positions of water depth measurements.

The total energy upstream the weir \( E_0 \) can be calculated as follows:

\[
E_0 = H_w + \frac{3}{2} Y_c
\]  

Where \( H_w \) height of weir, and \( Y_c \) critical depth above weir crest, which can be calculated as follows:

\[
Y_c = \sqrt{\frac{q^2}{2g}}
\]  

Where \( q \) discharge per unit width, and \( g \) gravitational acceleration. The total energy downstream the weir \( E_1 \) can be calculated as follows:

\[
E_1 = Y_1 + \alpha \frac{v_1^2}{2g}
\]  

Where \( Y_1 \) water depth at the toe of the weir, \( v_1 \) the mean velocity at the toe of the weir, and \( \alpha \) the correction coefficient of non-uniform velocity distribution.

According to Chow [8], \( \alpha = (1.03\text{-}1.36) \) for small channels. For simplicity, several researchers such as Salmasi, Chamani [1], Stephenson [2], Peyras, Royet [3], Kells [4], Chinnarasri, Donjadee [5] used \( \alpha = 1 \).
Due to flow conditions at the toe of weir (thin turbulence, and aerated flow), the measurement of flow depth is not accurate. Several researchers, such as Chinnarasri, Donjadee [5], Diez-Cascon, Blanco [9], Matos and Quinteia [10], Pegram, Officer [11], André and Schleiss [12], used the standard hydraulic jump formula to calculate the conjugate water depth as follows:

$$\frac{\Delta E}{E_0} = \frac{E_0 - E_1}{E_0}$$

(2)

Where ($\Delta E$) the energy dissipation on the weir ($\Delta E = E_0 - E_1$).

4. Results and analysis

Figure (4) shows the energy dissipation on a gabion stepped weir for downstream slopes 1:1 and 1:2. With increasing the discharge, dissipation of energy decreases for all downstream slopes and step configurations. This finding agrees with previous studies such as Salmasi, Chamani [1], Peyras, Royet [3], Kells [4], Chinnarasri, Donjadee [5]. At low discharges, when the flow is through-flow, the rate of energy dissipation was the maximum due to high resistance to flow offering by the voids of rock fill materials. The energy dissipation rate ranged between 85% and 89% for the through flow, similar to results found by Kells [4]. The effect of step face geometry was not appeared because the overflow was not started at this stage. At medium discharge (nappe and transition flow regimes), the effect of step face geometry increased. The inclined steps and end sill steps have higher energy dissipation than normal steps, might be due to the high resistance of step face. On the other hand, at high discharge (skimming flow), the effect of step face geometry decreased because the water skimmed over the pseudo-bottom formed by outer edge of steps, not on step face and the effect of step face geometry reduced.

The energy dissipation on gabion and impervious weirs for three step face geometries of normal, inclined, and end sill steps, and two downstream slopes of 1:1 and 1:2 is presented in Figure (5). It can be recognised that all curves had the same trend, where gabion weirs offered higher dissipation of energy than impervious weirs. At through-flow regime (low discharge) gabion weirs have more energy dissipation than impervious weirs within 17% for downstream slope 1:1 and 20% for downstream slope 1:2. With increasing the discharge the difference in energy dissipation between gabion and impervious weirs decreased because the effect of through-flow decreased. At the highest rates of flow (skimming flow regime), gabion have only 3% more energy dissipation than impervious weirs. Chinnarasri, Donjadee [5] stated that gabion weirs provided higher energy dissipation than impervious within 7% to 14% depending on downstream
slope. Nevertheless, Wuthrich and Chanson [6] presented a counterintuitive result, where they reported that gabion stepped weirs have the lowest rate of energy dissipation.

Figure 5. Energy dissipation on gabion and impervious weirs

Figure (6) illustrates the effect of downstream slope on energy dissipation efficiency.

Figure 6. Effect of downstream slope on energy dissipation in gabion weirs
The effect of the slope was the same for all step face configurations. The downstream slope had no effect on energy dissipation at low rate of flow (through-flow). At low discharges (through-flow regime), the downstream slope had no effect on energy dissipation, and the two slopes had the same efficiency, but the downstream slope effect appeared when the overflow regime started.

The energy dissipation increased as the downstream slope decreases for all step face geometries. The energy dissipation increased by 8% when the downstream slope decreased from 1:1 to 1:2. This due perhaps to the greater contact surface in a 1:2 slope than a 1:1 slope. This results have good agreement with previous studies such as Salmasi, Chamani [1], Peyras, Royet [3]. However, Kells [4] stated that the downstream slope has no clear effect on energy dissipation.

5. Conclusions
1. Energy dissipation efficiency decreases with increases in discharge for all downstream slopes and step configurations.
2. Gabion weirs have more energy dissipation than impervious weirs from 17%–20% for low rate of flow (through-flow regime) to 3% for high rate of flow (skimming flow regime).
3. At medium discharge (nappe flow and transition flow regimes), the effect of step face geometry was clear, and inclined steps and end sill steps offered 10% higher energy dissipation than normal steps.
4. Energy dissipation increased with decreases in the downstream slope for all step configurations, and the downstream slope of 1:2 had approximately 8% more energy dissipation than 1:1.

References


Collapse Prediction Using Static Compaction Curves for Untreated and Treated Basaltic Expansive Clays

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Keywords: Expansive clays, Lime stabilization, Collapse, compaction curves

This paper presents a new method in assessing collapse potential of an unsaturated compacted expansive clay as well as same expansive clay stabilized with lime. The widely used conventional compaction curves form the basis of a framework for such an assessment. The studied expansive clay was a residual soil derived from weathered Quaternary Basalt deposits located in the Victoria, Australia. For treated specimens, the optimum lime content was determined based on swell potential reduction. Static compaction technique was used to establish the compaction curves (virgin compaction surface) for untreated and lime treated soils. Different state paths were applied on both soils to investigate collapse potential under different conditions (moisture content, compaction and operational stresses). The test results showed that for an untreated or lime treated specimen compacted at a planned moisture content to a certain compaction stress, the collapse potential can be estimated if the specimen was wetted under different operational stresses. This estimation can be achieved using the virgin compaction surface.

1. Introduction

Collapsible soils are commonly characterized by sudden decrease in volume after wetting under a constant stress. The geotechnical engineers are challenged with the characteristic of collapse, prediction of degree of wetting and estimation of collapse settlement [1]. Prediction of structures collapse in the field relies on experimental laboratory results. The laboratory tests generally use 1-D Terzaghi theory of consolidation [2]. Many studies have investigated the mechanism of collapse under different stresses without much justification about the reasons for using these stresses [2-4]. Therefore, it is quite significant to further research and investigate factors affecting collapse behaviour considering moisture content and stress level especially for soils stabilized using chemical improvement techniques. Lime stabilization is widely used for reducing the collapse potential. Many studies have studied the importance of using lime as a binder to improve the properties of expansive soils, including a reduction in swelling and collapse potential. These studies investigated the suitability of lime in improving soil properties with majority of results obtained from specimens prepared at Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) [5-8]. At this condition, the behaviour of the dry side of the OMC and behaviour of the lime-stabilized soil under different operational stresses is not considered. Kodikara [9] presented the volumetric behaviour of unsaturated soils using moisture ratio (moisture content × specific gravity) as a main variable (instead of suction) as a more practical approach and described it in the Monash Peradeniya Kodikara (MPK) framework [9].
This paper presents the collapse behaviour of unsaturated untreated and lime treated clays. The expansive clay was treated with different lime contents. The optimum lime content (OLC) based on swelling potential was chosen in this study. Initially, the virgin compression surface proposed by Kodikara [9] was established for untreated and lime treated (at OLC) clays by developing the traditional compaction curves. Different state paths were applied to identify collapse paths after wetting under various stresses and then compared with collapse valued obtained from 1-D tests. The collapse potential was then predicted using the virgin compression surface.

2. Material
The selected expansive clay was a basaltic deposit located in the Victoria, Australia and classified as highly to extremely expansive clay [10]. The soil samples were collected at a depth of 1-2 m in plastic bags. The liquid limit, plastic limit, and plasticity index were measured and found to be 73.7%, 23.2%, and 50.5%, respectively.

3. Test Programme
Firstly, the OLC was obtained based on swelling potential and included standard Proctor compaction and 1-D swell tests. Secondly, the compaction curves were established to generate the virgin compression surface. Each curve was generated using static compaction technique. Thirdly, different state paths were applied to identify the collapse paths and consequently a proposed method to estimate the collapse potential using the virgin compression surface was obtain.

4. Test Results and Discussion
4.1. Optimum Lime Content (OLC)
The standard Proctor compaction test was performed according to ASTM-D698 [11] to measure the OMC and MDD for the untreated expansive soil specimens [12]. A series of 1-D swell tests were performed on the untreated and treated specimens. The specimens were compacted to the OMC and MDD using the Proctor compaction. The treated specimens were set to cure for 1, 7 and 28 days. Once the specimens were set up in an oedometer device, the specimens were floated with water under a surcharge of 25 kPa, simulating field stress conditions. The results suggest a significant decline in swelling when the specimens treated with 2% lime [12]. Furthermore, the swelling approached zero when the specimens treated with 4% lime with most of volume change occurring within 7 days of curing. Therefore, the OLC for this clay was found to be 4%. This percentage was used in this study.

4.2. Establishing of Compaction Curves
Compaction curves were established using static compaction technique starting from loosest state at nominal stress These curves repesents the virgin compaction surface proposed by Kodikara [9]. Any path on this surface can be achieved by compacting a specimen from nominal stress to a certain compaction stress or wetting or combination of loading and wetting. The specimens were prepared at moisture content varied from 0% to 50% and then compacted statically from the nominal stress of 2 kPa to different compaction stress. The static compaction stress varied from 2 kPa to 4000 kPa simulating a range of field conditions. The stress of 2 kPa resulted from the weight of the loading cap.
For untreated specimens, following to Australian Standard AS1289.5.1.1 [13], the dry soil was mixed with different amounts of water and left in a sealed bags for moisture content equilibrium. However, for treated specimens, dry soil was mixed with hydrated lime, and water was then added and mixed thoroughly to allow the cation exchange and flocculation processes to begin. The specimens were then compacted statically to different compaction stresses. On the dry side of the Line Of Optimums (LOO), the untreated and lime treated specimens were compacted at a stress rate of 20 kPa/min and 4 kPa/min, respectively, for a stress level less than 1000 kPa. While the rate increased to 100 kPa/min and 8 kPa/min for a stress level higher than 1000 kPa. On wet side of the LOO, the untreated and lime treated specimens were compacted at a stress rate of 0.5 kPa/min and 4 kPa/min, respectively, for a stress less than 1000 kPa. While the rates were 1.5 kPa/min and 8 kPa/min for a stress higher than 1000 kPa. At the end, the void ratio values were obtained. Fig. 1 shows the virgin compression surfaces for untreated and lime treated clays. To measure moisture ratio, it was important to measure the specific gravity of selected clay and it was found to be 2.71 [14].

4.3. State Path Series
After generating the virgin compression surface for both clays, a set of state path tests were carried out to study the suitability of the MPK framework. These tests included Loading/Wetting (LW), Loading/Unloading/Wetting (LUW), Collapse and swelling potential. Examples for these state paths are presented Tables 1 & 2. To perform the LW test, the calculated amount of water was added to the dry soil and then mixed thoroughly. After moisture content equilibrium (for untreated soil), the specimen was compacted statically from 2 kPa to a certain stress and then wetted under the compaction stress to the LOO. For treated soil, the specimens were cured for 7 days before wetting. The same procedure was followed to achieve LUW test except after loading to a certain stress, the specimens were unloaded to a stress less than the compaction stress and then wetted under the compaction stress to the LOO. For treated soil, the specimens were cured for 7 days before wetting. The same procedure was followed to achieve LUW test except after loading to a certain stress, the specimens were unloaded to a stress less than the compaction stress and then wetted under the LOO. For example, Fig. 2a presents the LW tests that were achieved on untreated specimens (LW 11 100 30.3, LW 15 500 21.8, LW 10 1000 19.5) and lime treated specimens (LW 10 100 36, LW 20 200 35.5, LW 10 1000 21.7) as shown in Fig. 2b. From Fig.2, it was evident that the specimens collapsed after wetting and followed the virgin compression surface when the specimens prepared at a degree of saturation ($S_r$) $> 37\%$ for the untreated specimens and 33\% for the treated specimens or wetted under compaction stress $> 1000$ kPa and $> 500$ kPa for the untreated and treated specimens, respectively. For LUW tests, the results showed that some specimens swelled after unloading to a certain operational stress and then wetting. For example, the untreated specimen LUW 10 1000 25 35.5 (Table 1) swelled after unloading to 25 kPa then wetting. It was clear that the loading path (from 2 kPa to 1000 kP) did follow the virgin compression surface and moved inside the surface during the
unloading (from 1000 kPa to 25 kPa). The specimen then swelled toward the stress contour for 25 kPa of the surface. However, other specimens collapsed after unloading to a cerain operational stress then wetting. For example, the untreated specimen LUW 20 300 200 30.3 (Table 1) collapsed after unloading to 200 kPa then wetting. Actually, the specimen swelled after wetting under the operational stress of 200 kPa, however the amount of swelling was very small and can be neglected, and the path of swelling hits the surface at a position before the LOO. Thus, by adding water beyond this point, the specimen collapsed and followed the contour of 200 kPa of the surface up to the LOO.

Figure 2. LW tests (a) untreated clay (b) lime treated (4%)

4.4. Collapse potential

A set of 1-D tests were performed on untreated and lime treated clays to investigate the collapse potential for specimens compacted at various moisture contents and compaction stresses then wetted under various operational stresses as shown in Table 2. The tests were divided into three groups based on compaction stress. The first group included specimens compacted to a stress of 200 kPa and then wetted under various operational stresses. However, group 2 and 3 included specimens compacted to stresses of 300 kPa and 1000 kPa, respectively and then wetted under various operational stresses.

5. Proposed Method to Predict Collapse Potential

A new method was suggested based on two facts. First, the state paths followed the virgin compression surface for specimens prepared at S_r > 37% for the untreated specimens and 33% for the treated specimens or wetted under compaction stress >1000 kPa and 500 kPa for the untreated and treated specimens, respectively. Therefore, the new method can be applied within this zone. Second fact is from the collapse potential results obtained from the 1-D tests. The new method relies on identifying the compaction stress and corresponded void ratio for a certain specimen. By recognizing these values, a constant void ratio line (LV) can be plotted on the surface. All points (specimens) generated by the intersection between the LV and stress contours will collapse after wetting. For example, the LV0.83 (Fig. 3) includes the stresses contours of 1000, 500, 400, and 300 kPa. All points generated by the intersection between the LV0.83 and the stress contours of 1000, 500, 400, and 300 kPa will collapse after wetting. The intersection points a, b, c and d are shown in Fig. 3. For example, specimen a, was prepared at 10% moisture content (e_w= 0.271) and compressed to 1000 kPa, collapsed after wetting to reach the LOO by following the aa’ line (Fig.3). Specimen b was prepared at 10% moisture content and compressed to 1000 kPa and then unloaded to 500 kPa and then wetted to the LOO. This specimen collapsed after continuous wetting by following the bb’ line in Fig.3. Note that bb’ is smaller than aa’. The collapse values obtained
from the proposed method was compared with that obtained from the 1-D tests (Table 2) and it was found that the results was too close.

![Figure 3. Proposed method to predict collapse potential](image)

<table>
<thead>
<tr>
<th>STATE PATH</th>
<th>DESCRIPTION OF THE TEST</th>
</tr>
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<tbody>
<tr>
<td><strong>LOADING-WETTING TESTS</strong></td>
<td></td>
</tr>
<tr>
<td>LW 10 1000 19.5</td>
<td>10% MC ((E_w=0.27)) LOADED TO 100 KPA, THEN WETTED TO 19.5% MC ((E_w=0.53)) U</td>
</tr>
<tr>
<td>LW 15 500 21.8</td>
<td>15% MC ((E_w=0.41)) LOADED TO 500 KPA, THEN WETTED TO 21.8% MC ((E_w=0.59))</td>
</tr>
<tr>
<td>LW 11 100 30.3</td>
<td>11% MC ((E_w=0.30)) LOADED TO 100 KPA, THEN WETTED TO 30.3% MC ((E_w=0.82))</td>
</tr>
<tr>
<td>LW 10 100 36</td>
<td>10% MC ((E_w=0.27)) LOADED TO 100 KPA, THEN WETTED TO 36% MC ((E_w=0.98)) T</td>
</tr>
<tr>
<td>LW 20 200 35.5</td>
<td>20% MC ((E_w=0.54)) LOADED TO 200 KPA, THEN WETTED TO 35.5% MC ((E_w=0.96))</td>
</tr>
<tr>
<td>LW 10 1000 21.7</td>
<td>10% MC ((E_w=0.27)) LOADED TO 100 KPA, THEN WETTED TO 21.7% MC ((E_w=0.59))</td>
</tr>
<tr>
<td><strong>LOADING-UNLOADING-WETTING TESTS</strong></td>
<td></td>
</tr>
<tr>
<td>LUW 10 1000 25 35.5</td>
<td>10% MC ((E_w=0.27)) LOADED TO 1000 KPA WHEN UNLOADED TO 25 KPA AND THEN WETTED TO 35.5% MC ((E_w=0.96))</td>
</tr>
<tr>
<td>LUW 20 300 200 30.3</td>
<td>20% MC ((E_w=0.54)) LOADED TO 300 KPA THEN UNLOADED TO 200 KPA AND THEN WETTED TO 30.3% MC ((E_w=0.82))</td>
</tr>
</tbody>
</table>

**MC**: MOISTURE CONTENT, **U**: UNTREATED CLAY, **T**: TREATED CLAY

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<thead>
<tr>
<th>GROUP</th>
<th>DESCRIPTION OF THE TEST</th>
<th>COLLAPSE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20% MC ((E_w=0.54)) LOADED TO 200 KPA, UNLOADED TO 100 KPA, WETTED TO SATURATION</td>
<td>9.2 (U), 2.4 (T)</td>
</tr>
<tr>
<td></td>
<td>20% MC ((E_w=0.54)) LOADED TO 200 KPA THEN WETTED TO SATURATION</td>
<td>13.8 (U), 8.8 (T)</td>
</tr>
</tbody>
</table>

Table 1. Summary of state path tests conducted on the untreated and lime treated clays
Table 2. Collapse potential obtained from 1-D tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Moisture Content</th>
<th>Stresses</th>
<th>Collapse Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>20% MC (E_w=0.54) LOADED TO 300 KPA, UNLOADED TO 100 KPA, WETTED TO SATURATION</td>
<td>2.6 (U)</td>
<td>8 (U), 1.4 (T)</td>
</tr>
<tr>
<td></td>
<td>20% MC (E_w=0.54) LOADED TO 300 KPA, WETTED TO SATURATION</td>
<td>10.1 (U), 4.1 (T)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20% MC (E_w=0.54) LOADED TO 300 KPA, WETTED TO SATURATION</td>
<td>12.1 (U), 5.5 (T)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20% MC (E_w=0.54) LOADED TO 500 KPA THEN WETTED TO SATURATION</td>
<td>2.1 (U), 1.5 (T)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>10%MC(E_w=0.27) LOADED TO 1000 KPA, UNLOADED TO 300 KPA, WETTED TO SATURATION</td>
<td>3.8 (U)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10%MC(E_w=0.27) LOADED TO 1000 KPA, UNLOADED TO 500 KPA, WETTED TO SATURATION</td>
<td>7.2 (U)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10%MC(E_w=0.27) LOADED TO 1000 KPA, UNLOADED TO 800 KPA, WETTED TO SATURATION</td>
<td>2.3 (T)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10% MC (E_w=0.27) LOADED TO 1000 KPA THEN WETTED TO SATURATION</td>
<td>10 (U), 3.2 (T)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10% MC (E_w=0.27) LOADED TO 2000 KPA THEN WETTED TO SATURATION</td>
<td>9 (U), 2.3 (T)</td>
<td></td>
</tr>
</tbody>
</table>

6. Conclusion

A series of laboratory tests were conducted to investigate the collapse potential of untreated and lime treated (4%) clays. The expansive clay selected was a basaltic soil located in Victoria, Australia. The collapse behaviour was investigated, using the conventional compaction curves, by applying LW and LUW tests on both clays. By comparing the results of these tests with the collapse results obtained from 1-D tests, a new method was proposed to predict collapse at different moisture contents, compaction stresses, and operational stresses.

References


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Development of strength model for CFRP-confined circular concrete columns affected by AAR

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Keywords: Design strength model, alkali aggregate reaction (AAR), carbon fibre reinforced polymer (CFRP), confinement efficiency, confinement ratio

Most theoretical models predicting the structural behaviour of carbon fibre reinforced polymer (CFRP) have been developed for normal concrete. In this study, thirty-two experimental results of tests on CFRP-confined circular columns of reactive concrete (damaged concrete by alkali-aggregate reaction (AAR) were employed to evaluate six existing strength models. Their performance significantly underestimated the true strength compared with the experimental results. Therefore, a simple strength models has been developed for reactive concrete, based on the experimental results for confined columns. Good agreement was found between the model prediction of the strength capacity of confined damaged concrete and the experimental values.

1. Introduction

Parallel to the experimental studies of the behaviour of FRP-confined concrete, a variety of theoretical models have been proposed to predict the behaviour using the stress-strain relationship. Most stress-strain models can be divided into two main categories, design-oriented models [1-4] and analysis-oriented models [5-6]. According to Ozbakkaloglu et al. [7], 88 stress-strain models had been proposed for FRP-confined circular concrete columns up to the end of 2011, comprising 59 analysis-oriented models, 13 design-oriented models and the remainder used either the plasticity approach [8-9], the extent of internal damage of the confined concrete core [10], or finite element models [11-13].

Design-oriented models demonstrate better performance than analysis-oriented models in predicting strain capacity and ultimate strength enhancement of FRP-confined concrete [7]. These authors explained that design-oriented models depend on the calibration of experimental tests in predicting equations which are very close to the behaviour of concrete confined with FRP under testing, whereas analysis-oriented models are dependent on expressions used for other models to simulate the behaviour. Most of the analysis-oriented models are based on force equilibrium and deformation compatibility at the interface between FRP and the concrete core [1, 7]. For example, Teng et al. [14] derived a model which can be applied to concretes of different strengths up to high values (110MPa) and for confinement materials other than FRP. These authors believed that analysis-oriented models are more powerful than design-oriented models, in spite of the latter models proposing a closer expression to reality. The powerful behind that as these authors explained due to the analysis-oriented models have considered the following features:
• The responses of FRP, concrete, and the interaction between the two materials.
• Bilinear stress-strain curves for both well- and weakly-confined concrete.
• Application of materials other than FRP.

Many numerical models have been proposed for the prediction of the stress-strain behaviour of concrete confined with FRP. The stress-strain prediction of some of these models depends on the results of testing of standard cylinders [2] or unreinforced specimens [3]. Furthermore, most of these models are based on normal concrete (undamaged). Few existing models are based on concrete damaged by fire [15] or AAR [16]. The present study employed the data from the authors’ work on testing 32 columns, including confined and unconfined, plain and reinforced, damaged by AAR in order to evaluate some current models and develop a new design strength model for confined concrete damaged by AAR.

2. Research Data

In developing the new design strength model, the results of testing the compressive strength capacity of 32 circular columns by the authors were employed. Some of these results have been published [17-19] and the others are still to be published. The columns are 204mm in diameter and 500mm in height, four being plain and 28 reinforced. One type of reactive concrete (damaged by AAR) was used to fabricate the columns. One type of high tensile strength carbon fibre CF (230/4900/400/50) (modulus of elasticity (GPa)/ tensile strength (MPa)/ weight (gm/m^2)/ sheet width (cm)) fabric was used as the confinement material with epoxy as the bonding material for wrapping the columns. Table 1 lists the details of the columns used in predicting the developed design-oriented strength model. As Table 1 shows, these columns were categorised into two series. Series A (C1-C6) and B (C7-C32) included plain and reinforced columns with reactive concrete (PCRc & RCRc), respectively. Series B included damaged concrete confined in the passive (C9-C14) and active stages (C15-C32) of AAR development. In active stage of confinement, the columns were wrapped by CFRP and the expansion due to AAR is still continuing, whereas in passive stage of confinement, the expansion due to AAR was approximately ceased when the columns are confined by CFRP. More details including expansion levels at confinement, test set up, mode of failure and stress-strain relationship can be found in [17-20].
### Table 1. Details of columns.

<table>
<thead>
<tr>
<th>Column number</th>
<th>Series</th>
<th>Description*</th>
<th>Expansion value (µs)**</th>
<th>Confinement condition</th>
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<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>C1-PC Rc-0</td>
<td></td>
<td>Unconfined</td>
</tr>
<tr>
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<tr>
<td>3</td>
<td>A</td>
<td>C3-PC Rc-1L</td>
<td></td>
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</tr>
<tr>
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<td>A</td>
<td>C4-PC Rc-1L</td>
<td>22521</td>
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<tr>
<td>5</td>
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<td>C5-PC Rc-2L</td>
<td></td>
<td>2L confined</td>
</tr>
<tr>
<td>6</td>
<td>A</td>
<td>C6-PC Rc-2L</td>
<td></td>
<td>2L confined</td>
</tr>
<tr>
<td>7</td>
<td>A</td>
<td>C7-RC Rc-0</td>
<td></td>
<td>Unconfined</td>
</tr>
<tr>
<td>8</td>
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<td>C8-RC Rc-0</td>
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<tr>
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<tr>
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<td>2L confined</td>
</tr>
<tr>
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<td>A</td>
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<td></td>
<td>2L confined</td>
</tr>
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<td>A</td>
<td>C15-RC Rc-1L</td>
<td>15820</td>
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</tr>
<tr>
<td>16</td>
<td>A</td>
<td>C16-RC Rc-1L</td>
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<tr>
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<td>A</td>
<td>C17-RC Rc-2L</td>
<td></td>
<td>2L confined</td>
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<tr>
<td>18</td>
<td>B</td>
<td>C18-RC Rc-2L***</td>
<td>***</td>
<td>2L confined</td>
</tr>
<tr>
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<td>B</td>
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<td>B</td>
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<td>B</td>
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</tr>
<tr>
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<td>B</td>
<td>C23-RC Rc-1L</td>
<td>10800</td>
<td>2L confined</td>
</tr>
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<td>24</td>
<td>B</td>
<td>C24-RC Rc-2L</td>
<td></td>
<td>2L confined</td>
</tr>
<tr>
<td>25</td>
<td>B</td>
<td>C25-RC Rc-1L</td>
<td></td>
<td>1L confined</td>
</tr>
<tr>
<td>26</td>
<td>B</td>
<td>C26-RC Rc-1L</td>
<td>9610</td>
<td>1L confined</td>
</tr>
<tr>
<td>27</td>
<td>B</td>
<td>C27-RC Rc-1L</td>
<td>9100</td>
<td>1L confined</td>
</tr>
<tr>
<td>28</td>
<td>B</td>
<td>C28-RC Rc-1L</td>
<td></td>
<td>1L confined</td>
</tr>
<tr>
<td>29</td>
<td>B</td>
<td>C29-RC Rc-2L</td>
<td>7695</td>
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</tr>
<tr>
<td>30</td>
<td>B</td>
<td>C30-RC Rc-2L</td>
<td></td>
<td>2L confined</td>
</tr>
<tr>
<td>31</td>
<td>B</td>
<td>C31-RC Rc-1L</td>
<td></td>
<td>1L confined</td>
</tr>
<tr>
<td>32</td>
<td>B</td>
<td>C32-RC Rc-1L</td>
<td></td>
<td>1L confined</td>
</tr>
</tbody>
</table>

* C = column, L = layer, P: plain, R = reinforced, Rc = reactive concrete
** Expansion value in radial direction [20]
*** C18-RC Rc-2L. This code means column number 18, a reinforced column with reactive concrete, confined with 2 layers of CFRP

### 3. Mechanism of Confinement

The use of uni-directional CFRP for the confinement of concrete represents a tri-axial state of stress resisting the lateral pressure produced by the dilation of concrete, and as a result the strength capacity of confined concrete is enhanced. The mechanism of confinement is based on the assumption of deformation...
compatibility at the interface between FRP and concrete. Concrete under compression behaves elastically until the compression stress reaches approximately 40% of ultimate failure stress and the relationship between lateral and axial strains is governed by the Poisson’s ratio. As stresses increase, the concrete cracks and more lateral expansion is developed by the concrete, leading to a high Poisson’s ratio [12]. The application of FRP produces confining pressure to resist the dilation effect of concrete and it behaves well compared to unconfined concrete [21]. As axial stress increases, the lateral strain also increases, due to the elasticity of FRP and failure occurs mostly due to rupture of the FRP [22-23].

4. Confining Pressure

A greater confinement effect can be achieved for structural members with circular cross-sections rather than rectangular, in which the effective confinement area is less, i.e., a high level of FRP efficiency and enhanced strength and strain capacities can be achieved by confining circular members [24-26]. As noted by Ozbakkaloglu et al. [7], a CFRP jacket produces a uniform pressure around the circumference of a circular column. The magnitude of the confining pressure depends mainly on the CFRP properties and the cross-sectional area of the column [27-28]. Based on CFRP properties measured previously by the authors [17] using coupon tests for one, two and three layers of CFRP, the ultimate confinement pressure (\( f_{lu} \)) can be calculated using Equation 1, and the results are shown in Table 2.

\[
 f_{lu} = \frac{k_a \rho_{frp} \varepsilon_{frp} E_{frp}}{2} = \frac{2t_{frp} \varepsilon_{frp} E_{frp}}{D} \tag{1}
\]

Where, \( k_a \) is the shape confinement coefficient which equals 1.0 for circular sections, \( \rho_{frp} \) is the volumetric ratio of FRP jacket, \( \varepsilon_{frp} \) is the ultimate tensile strain of FRP material, \( E_{frp} \) is the elastic modulus of FRP material, \( t_{frp} \) is the thickness of CFRP sheet and \( D \) is the diameter of the concrete core.

<table>
<thead>
<tr>
<th>No. of CFRP layers</th>
<th>( t_{frp} ) (mm)</th>
<th>( \varepsilon_{frp} ) (%)</th>
<th>( E_{frp} ) (GPa)</th>
<th>Diameter of column (mm)</th>
<th>( f_{lu} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.227</td>
<td>1.813</td>
<td>233</td>
<td>204</td>
<td>9.6</td>
</tr>
<tr>
<td>2</td>
<td>0.454</td>
<td>1.64</td>
<td>219</td>
<td>204</td>
<td>16.3</td>
</tr>
<tr>
<td>3</td>
<td>0.681</td>
<td>1.63</td>
<td>202</td>
<td>204</td>
<td>22.4</td>
</tr>
</tbody>
</table>

Table 2. Values of ultimate confining pressure with respect to different numbers of CFRP layers.

5. Effective Lateral Confining Stress (\( f_{le} \))

Most columns confined with CFRP fail due to CFRP circumferential (hoop) rupture. It is evident that for circular CFRP-confined columns, the ultimate circumferential strain (\( \varepsilon_{circumferential\,\,rupture} \)) at failure will be less than the tensile strain of FRP (\( \varepsilon_{frp} \)) measured by coupon testing, for several reasons [29-32]. Generally, the reasons are the non-uniform stress distribution in FRP jackets of cracked concrete, the curvature of FRP, the overlap, and others including load eccentricities, efficiency of work tester, and misalignment of fibres. Therefore, in developing a strength model, it is preferred to adopt the effective lateral confining stress, which depends on the measured hoop (circumferential) rupture strain of confined columns. Equation 2 is the formula used for calculating the effective lateral confining pressure (\( f_{le} \)):
Table 3 presents the calculated effective lateral confining pressures for all confined columns in series A and B. In determining $f_{le}$, the manufacturer’s data for the CFRP properties of thickness ($t_f = 0.227$ mm ply) and modulus of elasticity ($E_{frp} = 230$ GPa) were adopted, whereas the measured ultimate circumferential strain ($\varepsilon_{circumferential}$) in microstrain ($\mu$s) was based on the test results. The ultimate circumferential strain values were measured at the mid-height of the tested column, based on the results of the photogrammetry method employing digital image correlation technique (DICT).

Based on Table 3, the following conclusions can be drawn:

- The value of $f_{le}$ increases as the number of CFRP layers increases.
- The presence of steel ties appears to have no noticeable effect on the $f_{le}$ value compared to plain concrete. In other words, the steel ties have a marginal effect on $f_{le}$ values compared with the contribution of CFRP. A similar observation was made by [33].

6. Effect of Lateral Confining Stress on Strength Capacity of Confined Columns

The increase of uniform confining pressure provided by CFRP jacketing significantly enhances the ultimate strength capacity of confined columns compared to unconfined columns. The confinement efficiency values in terms of strength capacity ratio ($f_{cc}/f_{co}$) and confinement ratio ($f_{le}/f_{co}$) are given in Table 4 for confined columns from both series. Confinement efficiency represents the actual strength of a confined column compared with that of an unconfined column ($f_{cc}/f_{co}$) and the confinement ratio represents the effective lateral confining pressure of a confined column compared to the strength capacity of an unconfined column ($f_{le}/f_{co}$). The values of confinement efficiency and confinement ratio shown in Table 4 are presented graphically in Figs. 1, 2 and 3 for confined columns of series A, series B(passive confinement) and series B(active confinement), respectively. Generally, these figures indicate that the confinement efficiency value increases as the confinement ratio increases, i.e., a positive correlation can be observed between both values. The confinement ratios of confined columns for different series are more than 0.07, i.e., the columns are efficiently confined [2, 34].
<table>
<thead>
<tr>
<th>Column no.</th>
<th>Number of CFRP layer</th>
<th>( t_{frp} ) (mm)</th>
<th>( E_{frp} ) (GPa)</th>
<th>( \varepsilon_{circumferential} ) (hoop) (µs)</th>
<th>( D ) (mm)</th>
<th>( f_{le} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C3</td>
<td>1</td>
<td>0.227</td>
<td>230</td>
<td>13608</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>C4</td>
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<td>0.227</td>
<td>204</td>
<td>13479</td>
<td>7.0</td>
<td></td>
</tr>
<tr>
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<td>2</td>
<td>0.454</td>
<td>204</td>
<td>11135</td>
<td>11.8</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>2</td>
<td>0.454</td>
<td>11000</td>
<td>11000</td>
<td>11.5</td>
<td></td>
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<tr>
<td>C9</td>
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<td>0.227</td>
<td>204</td>
<td>9662</td>
<td>5.0</td>
<td></td>
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<td>204</td>
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</tr>
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<tr>
<td>C29</td>
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<td>C30</td>
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</tr>
<tr>
<td>C32</td>
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<td>0.227</td>
<td>204</td>
<td>7680</td>
<td>4.0</td>
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Table 3. Effective lateral confining pressure on confined columns for series A and B.
<table>
<thead>
<tr>
<th>Series number</th>
<th>Confined column number</th>
<th>$f_{cc} \circ (MPa)$</th>
<th>$f_{cc} / f_{co} \circ$</th>
<th>$f_{le} \circ (MPa)$</th>
<th>$f_{le} / f_{co} \circ$</th>
</tr>
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<tbody>
<tr>
<td>A 13.6</td>
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<td>5.82</td>
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<td>9.9</td>
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<td>C23</td>
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<td>9.9</td>
<td>0.497</td>
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<td>3.66</td>
<td>3.6</td>
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</tr>
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<td>7.3</td>
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<td>77.7</td>
<td>3.90</td>
<td>4</td>
<td>0.201</td>
</tr>
</tbody>
</table>

**Table 4.** Confinement efficiency values and ratios for confined columns of series A and B.
**Figure 1.** Relationship between confinement ratio and confinement efficiency for confined columns of series A.

**Figure 2.** Relationship between confinement ratio and confinement efficiency for confined columns of series B (passive confinement).
7. Strength Model

Most design-oriented models follow Equation 3, which was recommended by Richart et al. [21] to predict the ultimate strength of confined concrete based on confining pressure:

\[ f_{cc}' = f_{co}' + K_1 f_{le} \] (3)

This equation represents the relationship between confinement ratio \( f_{le}/f_{co}' \) and confinement efficiency \( f_{cc}' / f_{co}' \). Coefficient \( K_1 \) represents the gain of strength due to the confinement effect divided by the lateral confining pressure provided by the confinement material, as shown by Equation 2:

\[ K_1 = (f_{cc}' - f_{co}')/f_{le} \] (4)

To predict a strength model which is based on experimental results, the calculated values of \( K_1 \) for confined columns of series A and B are plotted against \( f_{le} \) in Figs. 4, respectively, to obtain one value of \( K_1 \) for each series and in combination. As Fig. 4 shows, for confined columns with reactive concrete, the increases in confining pressure reduce the value of \( K_1 \). To find one value of \( K_1 \) representing confined columns with reactive concrete, an analysis was carried out on combined data and the results are presented in Table 5:

\[ K_1 \text{ (confined columns with reactive concrete)} = 30.66 f_{le}^{-0.65} \]

By substituting \( K_1 \) values of confined columns for reactive concrete in Equation 1, Equation 5 was derived to predict the ultimate axial strength capacity of CFRP-confined columns for reactive concrete:
\[ f_{cc} = f_{co} + 30.66 f_{le}^{0.35} \] (5)

- Figure 4. \( K_1 \) versus \( f_{le} \) for confined columns with reactive concrete, (a) Series A, (b) Series B, (c) Series A & B (combination).

<table>
<thead>
<tr>
<th>Series no.</th>
<th>( K_1 )</th>
<th>Combined</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>( 14.62 f_{le}^{-0.38} )</td>
<td>30.66 ( f_{le}^{-0.65} )</td>
</tr>
<tr>
<td>B</td>
<td>( 28.15 f_{le}^{-0.59} )</td>
<td>( R^2 = 0.74 )</td>
</tr>
</tbody>
</table>

\( R^2 = 0.95 \)

\( R^2 = 0.69 \)

\( R^2 = 0.68 \)

Table 5. \( K_1 \) expression for confined columns with reactive concrete.

8. Performance of Existing Strength Models

A total of six published strength models for concrete confined by FRP were selected and reviewed to assess their accuracy compared with experimental results [1, 3-4, 6, 16 & 29]. Table 6 summarizes the selected models for FRP-confined concrete. Five of these models, excluding that of Abdullah [16], were developed...
for FRP-confined normal concrete, whereas the latter model was developed for FRP-confined reactive concrete. An analysis of these models for the prediction of the strength capacity of FRP-confined concrete and the percentage difference between the residual values (experimental data-predicted data) and the experimental results are presented in Table 7 for confined columns with reactive concrete. The comparisons between the predicted and experimental results of the strength of FRP-confined concrete for reactive concretes are also plotted graphically in Fig. 5. The following conclusions can be drawn concerning the performance of the selected models in predicting $f_{cc}^\prime$ and the comparison of the results with the experimental findings:

- All selected models, including Abdullah’s model [16], greatly under-estimated the predicted strength capacity of FRP-confined reactive concrete.
- Youssef et al.’s model [3] greatly under-estimates the strength of FRP-confined reactive concrete compared with the other models.
- The differences between the predicted strength $f_{cc}^\prime$ values and the experimental results varied from 2% over-estimated [16] to 67% under-estimated [3] for FRP-confined reactive concrete.
- All selected models performed well if the value of $f_l$ was used instead of $f_{le}$ in predicting $f_{cc}^\prime$.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Year</th>
<th>Model equation</th>
<th>Model type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toutanji [1]</td>
<td>1999</td>
<td>$f_{cc}^\prime = f_{co}^\prime (1 + 3.5 \frac{f_l}{f_{co}})^{0.85}$</td>
<td>Analysis-oriented</td>
</tr>
<tr>
<td>Teng et al. [6]</td>
<td>2007</td>
<td>$f_{cc}^\prime = f_{co}^\prime (1 + 3.5 \frac{f_l}{f_{co}})$</td>
<td>Analysis-oriented</td>
</tr>
<tr>
<td>Youssef et al. [3]</td>
<td>2007</td>
<td>$\frac{f_{cu}^\prime}{f_c} = 1 + 2.25 \left( \frac{f_{lu}}{f_c} \right)^5$</td>
<td>Analysis-oriented</td>
</tr>
<tr>
<td>Teng et al. [4]*</td>
<td>2009</td>
<td>$\frac{f_{cc}^\prime}{f_{co}} = 1 + 3.5 (\rho_k - 0.01)\rho_c$ ($\rho_k \geq 0.01$)</td>
<td>Design-oriented</td>
</tr>
<tr>
<td>Ozbakkaloglu and Lim [19]**</td>
<td>2013</td>
<td>$f_{cc}^\prime = c_1 f_{co}^\prime + k_1 (f_{lu}^a - f_{la})$</td>
<td>Design-oriented</td>
</tr>
<tr>
<td>Abdullah [12]</td>
<td>2013</td>
<td>$f_{cc}^\prime = f_{co}^\prime + 20.64 f_{le}^{0.28}$</td>
<td>Design-oriented</td>
</tr>
</tbody>
</table>

* $\rho_k$: confinement stiffness ratio, $\rho_c$: strain ratio  
** $c_1$: constant in the strength enhancement expression, $f_{lu}^a$: actual lateral confining pressure at ultimate

Table 6. Summaries of selected existing strength models.
<table>
<thead>
<tr>
<th>Col. no.</th>
<th>( f_{cc} ) predicted (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{cc} ) exp. (MPa)</td>
</tr>
<tr>
<td>C3</td>
<td>13.6</td>
</tr>
<tr>
<td>C4</td>
<td>66.3</td>
</tr>
<tr>
<td>C5</td>
<td>82.0</td>
</tr>
<tr>
<td>C6</td>
<td>79.0</td>
</tr>
<tr>
<td>C9</td>
<td>51.1</td>
</tr>
<tr>
<td>C10</td>
<td>61.3</td>
</tr>
<tr>
<td>C11</td>
<td>81.9</td>
</tr>
<tr>
<td>C12</td>
<td>78.2</td>
</tr>
<tr>
<td>C13</td>
<td>98.5</td>
</tr>
<tr>
<td>C14</td>
<td>100.0</td>
</tr>
<tr>
<td>C15</td>
<td>67.4</td>
</tr>
<tr>
<td>C16</td>
<td>71.6</td>
</tr>
<tr>
<td>C17</td>
<td>74.7</td>
</tr>
<tr>
<td>C18</td>
<td>95.3</td>
</tr>
<tr>
<td>C19</td>
<td>72.0</td>
</tr>
<tr>
<td>C20</td>
<td>68.0</td>
</tr>
<tr>
<td>C21</td>
<td>71.3</td>
</tr>
<tr>
<td>C22</td>
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<td>C25</td>
<td>71.5</td>
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<td>C26</td>
<td>74.1</td>
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<tr>
<td>C27</td>
<td>72.1</td>
</tr>
<tr>
<td>C28</td>
<td>73.0</td>
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<tr>
<td>C29</td>
<td>88.9</td>
</tr>
<tr>
<td>C30</td>
<td>95.6</td>
</tr>
<tr>
<td>C31</td>
<td>80.4</td>
</tr>
<tr>
<td>C32</td>
<td>77.7</td>
</tr>
</tbody>
</table>

Table 7. Application of selected existing models to comparison of predicted \( f_{cc} \) with the experimental results of confined columns with reactive concrete.
9. Performance of the Proposed Strength Models

The proposed Equations 5 for predicting the ultimate strength capacity ($f_{cc}'$) of confined reactive concrete columns were compared with the experimental data, and the results are presented in Table 8. The differences between the experimental results and predicted values of $f_{cc}'$ for confined reactive concrete columns vary with a ± 15% marginal error, with the exception of three results for C3 (-16%), C4 (-21%) and C9 (45%). The comparisons between the experimental test results and the predicted values according to the proposed strength models for confined reactive concrete columns are plotted in Fig. 6.

10. Conclusions

Six existing strength models were selected to evaluate their performance in predicting the strength of concrete confined by FRP in comparison with experimental results. The selected models significantly underestimated the true strength. Therefore, a new strength model has been developed for reactive concrete. The following conclusions can be drawn:

- Confinement efficiency ($f_{cc}'/f_{co}'$) increases as the confinement ratio ($f_{le}/f_{co}'$) increases for reactive concrete.
- Confinement ratios ($f_{le}/f_{co}'$) for all the tested confined columns with normal and reactive concrete are more than 0.07, which can be classified as within sufficient confinement.
- The gain of strength due to confinement ($K_1$) increases as $f_{le}$ decreases for confined reactive concrete.
- Simple strength model has been developed for reactive concrete, based on the experimental results for confined columns. Its performances in predicting the strength capacity of confined reactive concrete indicate good agreement compared with the experimental values.
<table>
<thead>
<tr>
<th>Series no.</th>
<th>$f_{co}$ (MPa)</th>
<th>Column no.</th>
<th>$f_{cc}^{(exp.)}$ (MPa)</th>
<th>$f_{cc}^{(pred.)}$ (MPa)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>13.6</td>
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<td>-16</td>
</tr>
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<td>61.1</td>
<td>74.2</td>
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<td>-5</td>
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<td></td>
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<td>69.1</td>
<td>4</td>
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<td>68.0</td>
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<td></td>
<td>C32</td>
<td>77.7</td>
<td>69.7</td>
<td>10</td>
</tr>
</tbody>
</table>

*Table 8.* Comparison of predicted proposed strength model results for confined circular columns with reactive concrete and the experimental results.
Figure 6. Comparison of experimental test results of $f_{cc'}$ and predicted proposed strength model values for confined reactive concrete.

Acknowledgements

The authors thank all the staff of the Smart Structures Laboratory at Swinburne University of Technology for their assistance and support. The first author would like to acknowledge the full scholarship support of the Ministry of Higher Education and Scientific Research of Iraq.

References


Geometry Effects on Energy Dissipation Over Stepped Spillway

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¹Deakin University, Victoria, Australia ²Babylon University, Babylon, Iraq

Keywords: Geometry, energy, dissipation, discharge, stepped, spillway.

The stepped spillway is a hydraulic structure, mostly used in dams. Normally, stepped spillways were designed to increase the energy dissipation level and hence decrease the risks at downstream. The current experimental investigation includes two groups of scale models, each with four different variables, to investigate the impact of step shape on flow characteristics and energy dissipation. The two groups were defined based on the downstream slope of the spillway (group one angle =26.6°, and group two angle = 21.8°). The test results with dimensional analyses confirms the significant influence of the geometrical parameters, namely, the height of step h, the radius of the end sill, and the number of steps N, on the energy loss process. Furthermore, at small values of discharge (at Nappe flow regime) the energy dissipation is the largest. Any increase in discharge will affect the flow regime and the energy dissipation, while any increase in the step number (N) will increase the energy dissipation.

1. Introduction

The stepped spillway is very old hydraulic structure. It has been utilized to control the flow rate and energy dissipation Chanson [6]. To decrease the risk of the damage in the structure and surrounding areas, The hydraulic structures must be designed to discharge all flowrates in a safe method Chanson [7]; Felder[13]. According to Carosi & Chanson [2] the advantages of stepped spillway design are higher energy dissipation rate, and smaller stilling basin size at downstream. The previous studies showed the geometry effect on energy dissipation.

Ideally, the high kinetic energy of flow that comes from water stored upstream of the hydraulic structures should be dissipated without any damage or erosion downstream. The traditional model of spillway designed during the 20th century has a flat slope (the chute is smooth), and uses a dissipation structure downstream to dissipate energy. However, stepped spillways have many useful features, e.g., their high energy dissipation allows decreased volume in the downstream stilling basin Toombes [21].

Felder & Chanson [12] described the energy loss and aeration processes for stepped spillways with moderate slopes. The outcomes indicated increases in the rate of re-aeration were related to increases in the rate of energy loss. Moreover, there are many functions have an impact on the energy dissipation in spillways such as the discharge, the structure slope, the structure and step geometry, and the number of steps Sorensen [20].

Chinnarasri & Wongwises [1] investigated the step geometry factors that have an influence on flow characteristics and the energy dissipation by using (horizontal steps, inclined steps, and steps with end sills). The outcomes noted to the relation between the energy dissipation and the critical depth in all the steps types. The performance of steps with end sills was the highest model in energy dissipation. Furthermore, the number of steps has clear effect (the energy dissipation increases when the steps number increases).
According to Hunt et al. [16] any increment in the step height will increase the energy dissipation. The energy dissipation is the difference between the total energy at upstream and downstream.

The study aim is to investigate energy dissipation and flow regimes in stepped weirs by using different steps shapes.

2. Flow regimes

Flow over stepped spillways is characterized by complex flow conditions, different flow regimes, and it is considered as a two-phase flow because of the high amount of air entrained with the flow. Thus, a physical model is necessary to study or design stepped spillways, the theoretical modelling may not represent the flow over the steps, despite a little effort to model the flow numerically Chen et al [9]; Carvalho and Martins [3].

Flow over stepped spillways is classified into three main flow regimes. These are nappe flow for relatively low discharges. This regime is the most efficient from the hydraulic viewpoint Pegram et al. [17], but it may not be practical for most of the cases. The transition flow regime, for relatively intermediate discharges. Chanson [8] recommended avoiding this regime in design because of strong hydrodynamic fluctuations associated with this regime. The third category is skimming flow for relatively large discharges. This regime is preferable due to its ability to convey larger amounts of flow with lower energy dissipation Frizell [14].

Nappe flow regime is defined as that regime where the flow passes from one step to another as a free-falling nappe with the maintenance of air pocket beneath. The falling nappe impinging on the step, with or without formation of the complete hydraulic jump. Fig. 2.2 is a schematic view of nappe flow regime over the stepped spillway. Nappe flow occurs for low relative discharges. It is induced by large step heights and flat slopes which are usually impractical Rajaratnam [19]; Peyras et al.[18]; Chanson [5]; Chamani and Rajartnam, [4].

Transition flow can be described as a water flow in a recirculation pool over the steps with significant water splashing and deflections Andre’ [1].

Skimming flow appears to move down the stepped face parallel to the pseudo bottom Pegram et al. [17]. Basically, in the skimming flow regime the flow skims as a coherent stream over the steps parallel to the pseudo bottom; the triangular space between the steps filled with recirculating vortices maintained by the transmission of shear force between the main stream and vortices Gonzales [15].

The characteristics of flow in term geometry effect were studied in current research.

3. Methodology and Experimental work

3.1. Testing facility

The study was at the civil lab in Deakin University, the facilities that used in the study are flume and point gages. The height of the flume equal to 25cm with 500cm long, and 7.5cm width. The flow rate range is 10 (l/min) to 150 (l/min). Figure (1) shows the flume. The sluice gate is at downstream to contrail the hydraulic jump location. The water tank capacity is (250 l). the weir location is 80mm from the inlet point. Three-
point gauges utilized to measure the flow depths at 200mm before the weir at upstream and the downstream point gauges location at 100mm and 3000mm from the toe.

2.2. Physical Models
Four physical models are used in experimental work to investigate the flow regimes and energy dissipation. In all the four models, the high is 300mm and width= 75mm with board crest= 50mm. The downstream slopes are (Group 1, $\theta= 26.6^\circ$, and Group 2, $\theta= 21.8^\circ$). Four different step geometries as shown in figures (2).

![Figure 1. The Flume scheme.](image1)

![Figure 2. The configurations of model ($\theta =26.6^\circ$).](image2)

3.3. Dimensional Analysis
All the studies on stepped weirs noted three groups of parameters have an influence on the weir performance in term energy dissipation. The three groups as follow:

1. The fluid properties
2. Flow characteristics.
3. The shape properties for spillway
\[
\frac{\Delta H}{H_0} = f\left(\frac{H_d}{y_c}, \frac{L}{y_c}, \frac{W}{y_c}, \frac{h_s}{y_c}, \frac{l_s}{y_c}, \frac{R_s}{y_c}, N_s, W_e, R_e, F_r\right) \quad \text{………………………………………………………………………………}(1)
\]

Where:

- \(H_d\): the height of spillway, the width (W), the step length (l_s), the step radius (R_s), number of steps (N_s), and the step height (h_s).

The utilized theory is Buckingham to derive the dimensional analysis. The parameters that used as repetitive parameters are \(y_c\), \(\rho\), and \(V\) because of they are available in most of the parameters. The outcome of the dimensional analysis is at equation 1.

### 3.4. Methodology

The main point in this study is to calculate the energy dissipation on the stepped weir. The upstream energy (Ho) is calculated by:

\[
H_o = Z_o + H_c = Z_o + \sqrt{\frac{q_w^2}{g}} \quad \text{………………………………………………………………………………}(2)
\]

\(H_c\) is the critical energy at critical flow depth, \(Z_o\) is the spillway height, \(g\) is the acceleration of gravitational and equal to 9.81 m/s^2. To find the downstream energy(Hd), equation 3 can be used.

\[
H_d = Z_1 + \frac{p}{\gamma} + \alpha \frac{V^2}{2g} \quad \text{………………………………………………………………………………}(3)
\]

\(Z_1\) is the elevation of the flume, \(\alpha\) is the correction coefficient of the kinetic.

According to Chow [11], measure the sequent depth of hydraulic jump at the toe of the spillway, where is the clear, non-aerated, water depth, then calculating, the upstream initial depth entering the jump, by the hydraulic jump equation:

\[
y_1 = \frac{y^2}{2} \left(\sqrt{1 + 8 \left(\frac{q_w^2}{\gamma y^2}\right)} - 1\right) \quad \text{………………………………………………………………………………}(4)
\]

And then can find the energy dissipation efficiency by using equation 5

\[
\% \frac{\Delta H}{H_0} = \left(\frac{H_o - H_d}{H_o}\right) \quad \text{………………………………………………………………………………}(5)
\]

\(\Delta H\) is the difference between the upstream energy and downstream energy on the stepped spillway.

### 4. Results and discussion

Both figures (3, and 4) show the relationship between the energy dissipation (\(\Delta H\)) and relative critical depth with step number and height (\(y_c/N_s\)). The models have same discharges and downstream angle.
(θ) with different step number and height for modified configurations (2, and 4). The energy dissipation increases when the step number increase. That means when the step number increase the flow path will longer and that will affect on flow characteristics and energy dissipation.

Figures (5 and 6) present the relationship between ∆H various the discharges. In figure (5), the angle (θ=26.6°) and high energy dissipation at modified configuration (2) with step number (N=10) was 88% at lower discharge (Nappe flow regime). However, the lower energy dissipation was in the traditional configuration (3) with step number (N=6) at height discharge (Skimming flow regime). Figure (6), for configurations with the angle (θ= 21.8°) shows the highest energy dissipation (∆H= 93%) at lower discharge (Nappe flow regime) in configuration (2).

From above, the influence of the step geometry and the turbulent flow for configuration (2) achieved better performance than the traditional configurations.
5. Conclusions

The conclusions from the outcomes are:

1. The observations for the traditional configurations have good agreement with the previous studies, such as the energy dissipation trend, and flow regimes boundaries.
2. The new step shape in configuration (2, and 4) achieved better performance in term energy dissipation and flow regimes
3. In all configurations with 10 steps give better performance than others with 6
4. steps at the nappe flow range, while, they have convergent results at
5. other flow regimes. In other words, step number impact is greater than step height at small discharges.

Acknowledgements

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References


Confining square cross section concrete column by Basalt textile impregnated by engineered cementitious composite

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Keywords: ECC, Basalt fibre, Confinement, sharp edges.

Confining concrete elements with fibre reinforced polymer (FRP) is proven to be an effective technique for improving the dilation and axial performance of concrete columns. However, its effectiveness is reduced significantly for non-circular columns. In addition, a few drawbacks of using FRP, such as brittleness of FRP sheet and poor performance of the material at high temperatures, have been found in the recently years.

This paper presents a feasibility investigation of a newly developed strengthening system, basalt fibre textile reinforced engineered cementitious composite (ECC). Three types of basalt continuous fibre grid were used, in combination with ECC to confine non-circular concrete columns. The experimental results revealed that the new strengthening system has significantly enhanced the load carrying capacity and ductility of non-circular concrete columns compared to the unconfined specimens and the textile reinforced mortars (TRM) technique. The results also have shown that ECC itself could be used as a new retrofitting material in column confinement.

1. Introduction

Confining concrete elements with fibre reinforced polymer (FRP) has been proven to be an efficient technique in improving the dilation and axial performance of concrete columns. However, if the cross section of the column has a non-circular shape, the efficiency of confinement reduced significantly [1, 2]. To improve the effectiveness of confinement, cross section modification techniques were considered as an effective technique in which the cross section shape was modified to a circular shape before being confined by FRP [3, 4]. However, the system has a few drawbacks, such as low resistance to fire and brittleness of FRP; potential hazards and permeability of the resin used. The insufficiency of vapour permeability caused concrete damage with the use of organic resins [5].

Fibre paste interaction and bond conditions of cementitious composite could be enhanced when textile fibre is used as an alternative material to FRP sheet [6]. This system includes textile fibre which is basically made from long knitted or non-knitted fibre impregnated in inorganic cement based mortar matrix. This combination results in an alternative strengthening system to FRP called textile reinforced mortar (TRM) [5]. However, the low tensile strength of mortar caused debonding failure of the system and the confinement was less effective than FRP jacketing due to such failure.

The innovation of ECC is an interesting inorganic based cementitious material which could be used for column confinement. ECC is a high performance fibre reinforced cementitious (FRC) material incorporating discrete PVA fibres with volume fraction of 2%. ECC is a unique type of cement mixture that exhibited
superior tensile strain-hardening compared to normal FRC with strain capacity in the range of 3-7% [7]. This paper presents a feasibility investigation of a newly developed strengthening system, basalt fibre textile reinforced ECC. Three types of basalt continuous fibre grid were used, in combination with ECC to confine non-circular concrete columns. The experimental results revealed that the new system has effectively enhanced the load carrying capacity and ductility of non-circular concrete columns compared to the TRM technique. The results also shown that, ECC itself could be used as a new retrofitting material in column confinement.

2. Experimental program

2.1. Material

The existing concrete columns were built from concrete batch provided by a local supplier (Easy Mix) and it is formulated for easy trowelling and finishing. The average 28 days compressive strength was 17 MPa. The material proportions of cement based mortar and ECC are listed in Table 1, same mix design was used except 2% PVA volume fraction of fibre was added to ECC mix. The strength of the mortar was 46 MPa with the density of 2096.3 kg/m³.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement</th>
<th>Sand</th>
<th>Fly ash</th>
<th>Water</th>
<th>Super plasticizer</th>
<th>PVA % volume fraction</th>
</tr>
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<tr>
<td>ECC</td>
<td>1</td>
<td>0.8</td>
<td>1.2</td>
<td>0.58</td>
<td>0.0055</td>
<td>2</td>
</tr>
<tr>
<td>Mortar</td>
<td>1</td>
<td>0.8</td>
<td>1.2</td>
<td>0.58</td>
<td>0.0055</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 1. Material proportions

The Basalt fibre textile was provided by the Jiangsu GMV New Material Science Ltd, China. Three different types of basalt fibres have been used: basalt grid 50*50mm; basalt grid 25*25mm and basalt mesh 10*10 mm. The mechanical properties of the basalt fibres which were provided by the manufacturer are shown in Table 2.

<table>
<thead>
<tr>
<th>Basalt type</th>
<th>Space mm</th>
<th>Density g/m²</th>
<th>Tensile strength N/mm²</th>
</tr>
</thead>
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<tr>
<td>Basalt mesh 10</td>
<td>10x10</td>
<td>125</td>
<td>2430</td>
</tr>
<tr>
<td>Basalt Grid 25</td>
<td>25x25</td>
<td>120</td>
<td>658.7</td>
</tr>
<tr>
<td>Basalt Grid 50</td>
<td>50x50</td>
<td>210</td>
<td>1166.2</td>
</tr>
</tbody>
</table>

Table 2. Material proportions

2.2 Confining Procedure and Design of Specimens

18 identical square plain concrete columns with a cross section area of 60*60mm² and a length of 200 mm were cast. All specimens were stored in curing pool for 14 days before cross section modification and confined by textile reinforced system. Standard cylindrical moulds with the diameter of 100mm and height of 200 mm were adopted to cast the confinement layer and convert the column cross section from square to circular shape. The square columns were installed vertically inside the mould and adjusted to become in the centre of the mould to get a uniform gap between the interior circumference of the mould and the column corner which was designed to have distance of 7.6mm (Fig 1(a)). The basalt reinforcement grids were made firstly and placed into the gap between the circular mould and the concrete column, and then the fresh ECC or mortar was poured with continuously shaking using a rubber hummer to eliminate any bubbles in the matrix as shown in Fig 1(b). The specimens were demoulded and stored in curing pool until
being tested after 28 days. The specimens were classified into six groups, three samples for each group and the group details are shown in Table 3. For each short circular confined column, four strain gauges were mounted at mid height. Two gauges were aligned along axial direction $180^\circ$ aside, and two gauges along hoop direction aligned and located $180^\circ$ apart to record axial and hoop strain respectively. The specimens were subjected to axial compression load using MTS machine with a constant load rate of 0.2 MPa/s according to ASTM C39.

3. Experimental results and discussion

The results of average axial strength from three tested specimens of each group are listed in Table 3.

![Image](image1.png)

**Figure 1.** Confine ment technique.

<table>
<thead>
<tr>
<th>Group</th>
<th>Confinement</th>
<th>$\varepsilon_{co}$ (µm/m)</th>
<th>$\varepsilon_{cc}$ (µm/m)</th>
<th>$f_{co}$ (MPa)</th>
<th>$f_{cc}$ (MPa)</th>
<th>$f_{cc}/f_{co}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Control sample</td>
<td></td>
<td>576</td>
<td></td>
<td>20.2</td>
<td></td>
</tr>
<tr>
<td>C-M25</td>
<td>Mortar+basalt25</td>
<td></td>
<td></td>
<td>1067.2</td>
<td>20.2</td>
<td>28.54</td>
</tr>
<tr>
<td>C-E</td>
<td>ECC</td>
<td></td>
<td>576</td>
<td>3090.9</td>
<td>20.2</td>
<td>38.9</td>
</tr>
<tr>
<td>C-E25</td>
<td>ECC + basalt25</td>
<td></td>
<td>576</td>
<td>2807.1</td>
<td>20.2</td>
<td>35.81</td>
</tr>
<tr>
<td>C-E50</td>
<td>ECC + basalt50</td>
<td></td>
<td>576</td>
<td>2117.9</td>
<td>20.2</td>
<td>32.145</td>
</tr>
<tr>
<td>C-E10</td>
<td>ECC + basalt10</td>
<td></td>
<td>576</td>
<td>2662.1</td>
<td>20.2</td>
<td>31.18</td>
</tr>
</tbody>
</table>

**Table 3.** Average confinement effectiveness results of tested specimens

3.1. Confinement effectiveness

$f_{co}$ and $f_{cc}$ are the ultimate axial strength of unconfined and confined specimens respectively; $\varepsilon_{co}$ and $\varepsilon_{cc}$ are the unconfined and confined axial strain at the corresponding ultimate axial strength respectively; the ratio $f_{cc}/f_{co}$ is the confinement effectiveness of the confined concrete square column.

Table 3 indicates that confinement effectiveness of columns confined by textile reinforced mortar group (C-M25) was 1.41; whereas, the effectiveness ratio was 1.93 for columns confined with ECC material, which is the highest effectiveness recorded in this experimental study. The average confinement effectiveness was found to be 1.77, 1.59, and 1.54 for group C-E25, C-E50, and C-E10 respectively. These results indicate that using ECC material alone, leads to a higher confinement effectiveness level of confined square columns. Considering ECC material in a combination with basalt fibre textile, basalt fibre with 25 mm spacing (group C-E25) had a significant enhancement on the confinement effectiveness and a considerable strain capacity compared with group C-M25 where the same basalt fibre textile was applied.
with normal cement mortar. C-M25 also had much less ultimate strain compared with C-E25 due to the effect of PVA fibre in the confinement layer of group C-E25.

3.2 Stress-strain behaviour
Axial stress strain diagrams for each group are compared in Fig 2. As shown in Fig 2, the control and TRM groups followed the typical bilinear behaviour as reported in the literature [5]. The diagrams with ECC confinement are characterized by a short ascending branch, followed by a large strain hardening region, before suddenly dropped until failure. This notable difference in the stress strain diagrams was attributed to the different performance of mortar and ECC. Due to the low tensile strength of mortar and the subsequent debonding failure, the high tensile strength of basalt fibre could not be fully utilized, indicating the effect of confinement is limited for these samples. For ECC confined samples, when the load increased, more multi-cracks were continuously formed, but debonding failure was prevented due to strain hardening of the material. The effect of confinement was largely increased.

3.3. Failure modes
For the specimens confined by TRM (C-M25), crack initiated in the mid-height of the specimens and progressed to the two ends. Debonding failure was noticed when mortar reached its tensile strength. Ductile failure was still observed for this group compared to the brittle failure of the control group, this was due to basalt fibre carried out large increased hoop stress until it ruptured when mortar failed. For the specimens confined by basalt fibre and ECC material or just ECC, the failure modes were quite different. At mid-height of each specimen, very tiny multi-cracks were observed when mortar reached its tensile strength, the cracks propagated to the ends of each specimen. The failure was due to fracture of basalt fibres not debonding of ECC. The failure pattern was very ductile compared to TRM group. Figures 3 shows the failure patterns of the tested groups.

4. Conclusion
The strengthening techniques proposed in this paper were considered to strengthen the existing square concrete column with different types of confinement. The experimental results indicated that ECC or a combination of ECC with basalt fibre textile, as a confining layer, can effectively increase the axial load capacity and ductility of square concrete columns compared to the unconfined and TRM specimens. The less effectiveness of TRM technique was due to debonding failure of mortar where the basalt fibre textile reinforced ECC technique resulted in a fibre fracture failure of confined specimens.
References


CFRP-strengthening of steel plates in multiaxial fatigue loading

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Keywords: Fatigue, Mixed-mode loading, CFRP, Strengthening, Steel plates,

Ensuring the structural integrity of steel bridges and railway roads is the ever-existing challenge that faces Civil engineers. Most of these structures are prone to fatigue failure due to the constant changing in their operational conditions over the increasing service life. Experimental research showed that strengthening steel components with a composite material such as carbon-fibre-reinforced-polymer (CFRP) can significantly upgrade their capacity and prolong their service life in tension fatigue loading. However, steel components of complex details in old bridges mostly fail under more complex fatigue loading. In this paper, a summary of the ongoing experimental study on the CFRP-strengthening of steel elements under multiaxial loading is presented. The performance of steel plates with different damage levels, under a wide range of the mode-mixity ratio, and strengthened with CFRP materials of different mechanical properties were investigated. Based on the experimental results, the fatigue behaviour of the CFRP strengthening system of steel plates in mixed-mode fatigue loading is approximated by the fatigue behaviour of the same strengthening systems in the tension fatigue loading by using a mixed-mode modification factor (MMMF) and the superposition principle. The MMMF was driven from all the test data and will be employed to develop a design guidance for engineers who interest in the maintenance of ageing bridges. The future work will focus on developing a more reliable strengthening systems for structural elements experience a complex fatigue loading.

1. Introduction

Fatigue failure is the most common threat to the structural integrity of steel bridges. Complex details and steel connection in ageing steel bridges are prone to fatigue failure and strengthening these components poses a precaution action to ensure the structure functionality. Composite material such as carbon-reinforced-polymer shows a great alternative and a sustainable solution in strengthening steel elements compared to traditional construction materials [1, 2]. It is almost two decades since the first application of the CFRP on steel bridge [3]. Although, several design guidelines have been introduced regards employing the composite material in structural engineering [4, 5], the innovation material still in its early stage
compared to the traditional construction material. Due to a variety of the properties of the composite material, different methods were developed to utilizing them in strengthening steel structures. The CFRP material is either adhesively bonded to the surface of the steel components or attached to the strengthened elements after being prestressed. The prestressed CFRP is either bonded or unbounded to the strengthened elements. Strengthening with prestressed bonded CFRP composites usually requires more efforts and preparation, compared to the strengthening with non-prestresses bonded composite [1], therefore more attention was paid to the latter. However, the advantage of the CFRP strengthening in prolonging the fatigue life of steel plates strengthened with prestressed composite is almost always higher than the benefits of the strengthening with non-prestressed CFRP.

Several studies were conducted to investigate the capabilities of the new composite material in steel structures application [7-14]. It is revealed that the efficiency of the CFRP strengthening can be increased by increasing of the in-plane stiffens of the CFRP strengthening system. The improvement in tension fatigue strengthening was presented in terms of increasing the number of the CFRP sheet layers [7], the CFRP mechanical properties [8], the strengthening scheme [9], and the strengthening technique [10]. For example, Wu et al. [11] found that employing ultra-high modulus of the CFRP plates in retrofitting steel structures showed superiority in delaying crack growth initiation. Furthermore, it revealed that by prestressing CFRP strips before being applied to the repaired members the fatigue life can be extended further and crack propagation can be arrested [12]. Adhesively bonded CFRP sheets were utilized recently in the strengthening of West Gate Bridge in Melbourne, Australia, presenting one of the largest applications of the CFRP strengthening of the bridge in the world [13]. Similarly, prestressed CFRP laminates were used in fatigue strengthening of a 120-year-old railway riveted metallic bridge in Switzerland [14].

The current knowledge of the CFRP application in steel structures [15] is almost limited to the tension fatigue strengthening and it developed based on the assumption that the fatigue crack exhibits tensile stresses only. However, in fact, the cracks in complex details in ageing steel bridge are often exist randomly oriented to loading applied and therefore, a state of combined action of stresses could be developed at the crack tips before propagates. The fatigue performance of the CFRP strengthening under such complex load conditions is not clear and need to be investigated.

In this context, the authors of the present paper have investigated the fatigue performance of the CFRP-strengthened steel plates with cracks oriented between (10°-90°) and crack length ratio (slit length to the plate width ratio) of 2% [16]. Test results revealed that mixed-mode crack propagation curves were shifted to mode-I crack growth. The amount of shifting was proportional to the ratio of shear to tensile stresses at the crack tip, i.e.; the smaller the crack angel, the larger the shift. This work was followed by investigating the damage level effect on the mixed-mode fatigue behaviour of the CFRP strengthening system. In this study, CFRP strengthening of steel plates with the level of damage of 10% and 30% were covered [17]. In the present paper, a summary of the main outcomes of the experimental results of the CFRP strengthening of steel plates with inclined cracks is presented.

2. Experiments

2.1. Test specimens

All the test specimens were designed with identical geometry and dimensions as illustrated in Figure 1. Steel plates were notched with a 5mm central hole representing the bolted connection in steel bridges. Then, to develop a state of mixed-mode loading (I+II) at the crack tip, and by using the electric discharge
machine, the central cracks were oriented between (10°-90°). The length of the crack starter was changed to represent damage levels of 2%, 10%, and 30. Two strengthening schemes were selected in this study; namely configurations A (the CFRP composite covered the whole crack area), and D (the CFRP composite positioned away from crack tips) as shown in Figure 1, based on the output of previous work in Monash University [11]. The strengthening stiffness of scheme A is more than twice that of scheme D.

The bondline represents the weakest part in the CFRP-steel strengthening system and therefore the bonding strength is a crucial factor in determining the efficiency of the strengthening system [18]. To provide a clean, and chemically-active surfaces of the adherends, the bonded steel surface was abraded first by the sand-blasting method, then cleaned by using Acetone solvent, before applying a uniform adhesive layer of the Araldite 420 on both substrates of the CFRP and steel plates. The air voids and the excess adhesive were drained away under the weight of steel block was put on top of the strengthened specimens.

![Figure 1](image)

**Figure 1.** Geometry and dimensions of the CFRP-strengthened specimens

(a) fully-strengthened specimen (scheme A), (b) partially-strengthened specimen, (scheme D), (c) notch details, and (d) Single-sided repair methods (in mm, not to scale)

### 2.2. Material Properties

The fatigue tests were conducted on CFRP adhesively-bonded steel plates with central cracks. The mechanical properties of the test specimen’s components are presented in Table 1. The tensile coupon tests of the 300Plus grade of steel plates were conducted according to the AS1391 [20]. Different mechanical properties of the composite material were utilized in the current work for comparison purpose among a range of the strengthening stiffness. The composite material provided either in sheets or
laminates and both utilized in the strengthening system of steel plates with inclined crack. The CFRP laminates included the unidirectional pre-cured MBRACE laminates 460/1500, and S&P Laminates CKF 200/2000 with cross sections of 50mm x 1.46mm, and 50mm x 1.4mm, respectively. In this paper it is referred ultra-high-modulus campsite UHM-CFRP to the former and normal modulus composite NM-CFRP to the latter. The steel and the CFRP component were bonded by using Araldite 420. The test results of strengthening with sheet campsite is undergoing and therefore it has not presented herein.

The manufacturer datasheet

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
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<td>2500</td>
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<td>-</td>
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</tr>
<tr>
<td>Thickness (mm)</td>
<td>10</td>
<td>1.46</td>
<td>1.4</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\)The manufacturer datasheet

Table 1. Material properties of test specimens

2.3. Fatigue Loading

All the CFRP bonded steel plates were tested in the Civil Engineering Laboratory at Monash University in Melbourne, Australia using Instron 8802 servo-hydraulic testing machine, as shown in Figure 2. The test specimens were subjected to uniaxial fatigue loading with constant amplitude and 20Hz frequency. The sinusoidal cyclic load fluctuated between 150MPa, and 15MPa, with a maximum stress level at the steel cross-section constitutes nearly 46% of the steel tensile yield stress. The test loading level increased to 60% of the steel tensile yield stress, for specimens did not fail under the designed load. For the measurement of the crack length on the fracture surface, the beach marking method was used, see Figure 3. For this purpose, the stress range changed so that the stress intensity factor at the crack tip changes in a way to introduce visible marks at the surface of fracture. The crack length was measured as reported in the preliminary work of the authors [16].
3. Experimental Results and Discussion

3.1. Fatigue Life

The fatigue life represents the number of stress cycles that the specimens sustained before cracks propagated along the plate width. The test results of the conducted tests so far were summarized and discussed in the earlier work of the authors in [16, 17]. To summary, the main output will be highlighted here as follows:

A. The fatigue test for specimens with crack angles of 10° and 30° with a crack length ratio of 2% was run out. This explained by the significant drop in the tensile stress intensity factor at the crack tip due to the crack orientation. Under higher test loading of 60% the tensile yield stress of the steel, specimens with the crack angle of 30° were failed whereas that of the crack angle of 10° did not fail. The stress magnitude of the former was less than the fatigue endurance limit of the tested steel even with higher test loading.

B. For the same crack angle with higher damage level of 10% was also run out under the test loading whereas failed under the increased fatigue loading. The increase in the damage level came with an increase in tensile stress intensity factor was adequate to initiate the damage and propagate the crack.

C. The fatigue life increased proportionally with the increased stiffness of the CFRP strengthening system, i.e.; the higher Young’s modulus of the composite material the more advantage of the CFRP strengthening, and then the longer fatigue life.

D. The strengthening scheme has a noticeable effect on the fatigue life of the CFRP-strengthened plates with inclined crack. For specimen with the same crack angle, the fatigue life of specimens fully covered with CFRP composites extended higher than that of specimens partially covered with the composite material.

E. For specimens with higher damage level, the fatigue life increase was higher. This trend change when the effect of the initial crack angle included, i.e.; for specimens with longer crack length 30%, the number of cycles the crack takes to propagate from 2% to 30% were extracted from a-n curve of 2% case and was counted in the total fatigue life of the specimen with longer crack length. The fatigue
performance of the CFRP-strengthened specimens with different crack angles highlights the importance of the earlier repair of the notched plates.

F. The shifting phenomenon observed in mixed-mode crack propagation curves steel plates strengthened with different mechanical properties, i.e.; CFRP with Ultra-high-Young’s modulus vs. CFRP with normal Young’s modulus.

G. The difference in the shifting amount (number of cycles in mixed mode, \( Nm \)) between strengthening with normal modulus composite NM-CFRP and ultra-high-modulus composite UHM-CFRP was less than 23% for specimens with crack angle of 30° and smallest damage level. This could be explained by the effect of the strengthening system on the mode-mixity ratio (Shear stress to tensile stress intensity factors S-SIF/T-SIF). The CFRP with higher stiffness has higher effect in terms of decreasing the T-SIF, and then results in higher ratio of mode-mixity. Consequently, the shift amount \( Nm \) increases as the stiffness of the strengthening system improved (UHM-CFRP over NM-CFRP, and configuration A over D). It is obvious that the amount of shift is related to the mode-mixity and the strengthening stiffness ratio.

4. Crack Propagation in Mixed-Mode Loading (I+II)

The test results showed the positive effect of the crack inclination on the fatigue life. There was a proportional increase in the fatigue life to the decrease in the crack/loading angles. This trend can be related to the state of the local stresses at the crack tip. For different strengthening schemes, and also the unstrengthened specimens, the fatigue life increase was almost consistent for each crack angle. In mixed-mode crack propagation, the crack first initiates under mixed-mode loading before kinked and propagate in pure tensile mode. This definition was employed to identify the fatigue life of steel plates with inclined cracks by deriving modification factor relates the fatigue life of steel plates with inclined crack with that of plates with normal flat crack for different damage level and strengthening schemes. Figure 4 shows the shifting in crack propagation curves of steel plates with inclined cracks from the curve of the same specimens with flat crack. The mixed-mode modification factor is a function of the crack angel and it represents the ratio of the number of cycles under mixed-Mode condition \( \text{(Nm)} \) to that under mode I. Figure 5 shows the mixed-mode modification factor derived for a range of the tested specimens. The tested data was approximated linearly (straight lines) and also approximated by using nonlinear fitting (the curves in Figure 5).

![Figure 4](image1.png)

**Figure 4.** Schematic view of the shifting phenomenon in mixed mode fatigue crack growth [13] curves [16].
5. Conclusions

Fatigue tests were conducted on CFRP adhesively-bonded steel plates with inclined cracks. Steel plates were notched with six crack angles to simulate the case of mixed-mode (I+II) loading at the crack tip. The crack length was introduced in steel plates in different ratios to the plate width to highlight the effect of applying the CFRP composites at various stages of the service life of the steel components. The following conclusions were drawn based on the available test results:

1. The fatigue test for specimens with smallest damage level and crack angles of 10° was run out. The stress intensity factor at the crack tip was less than the endurance limit of the steel plates even when the test loading was 60% of the tensile yield stress of the steel plates.

2. The fatigue life increased proportionally to the decrease in crack angle due to the reduction in the crack driving force the tensile stress intensity factor at the crack tip. This increase was presented as a shift in the crack propagation curve. Consequently, the mixed-mode crack propagation curves were approximated by the same curve in pure tensile loading (mode I).

3. The shifting observed in the mixed-mode crack propagation curves of steel plates strengthened with UHM-CRRP still exists for steel plates strengthened with NM-CFRP. Therefore, mixed-mode crack propagation curves of CFRP-strengthened steel plates could be approximated by the same curve of mode-I whatever the strengthening stiffness.

4. Based on the shifting observed in the mixed-mode crack propagation curves, a modification factor was derived to and employed in predicting the fatigue life of CFRP strengthened steel plates with inclined cracks. The total fatigue life prediction requires only the fatigue life of the steel plates with flat crack and the modification factor. The modification factor was presented for steel plates with inclined cracks, different damage level, and two strengthening schemes.

5. The advantage from the CFRP strengthening in prolonging the fatigue life was proportional to the strengthening scheme, and the improvement in the mechanical properties of the composite material.

6. For strengthening systems with different properties of the composite material, the amount of shift is related to the mode-mixity ratio and the strengthening stiffness ratio.
Acknowledgements

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References


Design of Concrete Structures - Spreadsheet Development for The Design of Concrete Columns

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The main aim of this paper is to develop a design tool that helps structural engineers with the design of reinforced concrete columns and to mainly generate interaction diagrams, which gives an indication whether the column section is able to withstand the loads it is subjected to or not. In order to develop the discussed tool, there are few objectives that were achieved in the literature review and in the tool itself. These objectives include; discussing the material properties; design philosophy and methods of design according to the Australian standard for concrete structures (AS3600) and the American code (ACI-318); identifying current design tools and illustrating their features; and developing the discussed tool and validating it to other reliable tools commercially available. The developed spreadsheet analyses rectangular and circular reinforced concrete column sections under uniaxial loading in two unit systems, SI, and Imperial.

1. Introduction

This research paper deals with the design of reinforced concrete structural elements. It mainly focuses on the design of concrete columns. Concrete is a mixture of sand, gravel and aggregates that are held together by a cement paste and water, concrete was first used for structural purposes in 1832, according to McCormac, J. & Brown, R. [10]. Reinforced concrete can be defined as a combination of concrete and steel, where steel provides tensile strength lacking in the concrete and concrete provides compressive strength lacking in the steel [4]. Reinforced concrete is considered to be the most important material available for construction in the 21st century. It is used almost in every structure, such as, buildings, bridges, dams, tunnels, retaining walls, tanks, pavements and culverts.

There are many programs and tools used to assist structural engineers with the design of structures or structural elements. This research paper will compare between the design methodologies of reinforced concrete columns used in Australia in accordance to the Australian standards AS3600 and in Iraq according to the American standards ACI 318-11 since it is used in Iraq. Moreover, this paper will also compare between the materials used in Australia and Iraq in terms of their properties and grades.

The results of the comparison were used to develop a spreadsheet called OK.Column that assists structural engineers with the design of typical concrete columns. The developed tool will adopt flexible features and a friendly interactive interface, for example it will be flexible to the different grades of concrete and steel as well as to the types and shapes of typical concrete columns. Furthermore, the main objective of OK.Column is to generate interaction diagrams for reinforced concrete columns. Moreover, the discussed tool will have a section that validate its results to other reliable tools. The design will be based on the Australian standard AS3600 and the American standard ACI 318-11.
2. Literature Review

The literature review discusses few topics. These topics are: the properties of concrete and reinforcement steel, the methods of design according to the Australian code AS3600 and the American code ACI318, the features of other available tools.

2.1. Materials Properties

To design a reinforced concrete column according to a certain standard, it is important to identify the materials involved in that design. Therefore, this paper will deal with the requirements for the reinforcement steel and the concrete.

2.1.1. Concrete properties

There are a number of standard grades of concrete in Australia that are usually ordered premixed. These grades are: 20 MPa, 25 MPa, 32 MPa, 40 MPa, 50 MPa, 65 MPa, 80 MPa, 100 MPa, Standard grade concrete is usually delivered with a maximum aggregates size of 20mm [3].

2.1.2. Reinforcement Steel

All reinforcement steel used in Australia must comply with the specification stated in AS4671. Most important aspect is the grades allowed in Australia, and the code states that the following grades are the standard in Australia and New Zealand: 250N, 300E, 500L, 500N, 500E, where the number refers to the yield stress.

*note: the properties of materials used in Iraq are not mentioned and this is due to the lack of information available online.

2.4. Design Method

This section of the literature review covers the design method of typical reinforced concrete columns based on the Australian standard AS3600 and American standard ACI318. This section will cover the design of symmetric reinforced concrete columns with rectangular and circular sections of normal strength concrete and high strength concrete.

Research has shown that the methods of design in the American and Australian code do not differ from each other much, the differences come from the reduction safety factors used in each code and the unit system used in each code. Therefore, the results of the interaction diagrams of both design code should be close.

The developed interaction diagrams by the Australian and the American codes depend on the grades of both concrete and steel. These diagrams are generated for different column sections (Rectangular and circular) and sizes. Moreover, the developed diagrams depend on the distance between the center of the bars on one side of the column to the center of the bars on the other side of the column divided by the column depth, this value has different symbols (γ gamma in the American code, g in the Australian code). The interaction charts are drawn by adopting few modifications on the moment and axial load equations [8]. The adopted modifications by the Australian standards and the American standards are illustrated thoroughly below. The table below compares between the American standards and the Australian standards in term of their equations and the symbols used in each equation.
3. Modeling

For the purpose of developing interaction diagrams for concrete columns a spreadsheet was developed. This spreadsheet works with two codes and two units systems. And will deal with the design of circular and rectangular cross sections. Furthermore, the effect of spiral confinement is also taken into consideration.
This design tool models the concrete column cross-section as multiple layers of reinforcing steel, and uses an iteration approach to calculate the strain in each layer, and how the neutral axis position change with each interval in the iteration process [6]. By tabulating the results obtained from the iteration process it is easy to develop the interaction diagram using the insert graph function available in Microsoft excel.

The above mentioned iteration process is carried over by using the readily available excel functions as well as hand written more specialized functions using visual basic accessed from the developer section [5].

All the mathematical calculations and formulas are based on what is introduced in the two codes (ACI 318, AS-3600). Furthermore, some of the formulas are taken from text books reviewed during the first part of this project.

4. Results and Discussion

After building the OK.Column, the results were compared to another tools results in order to validate it. The interaction diagram shown in Fig. 1 below is for a rectangular section with the properties shown in the same figure.

As shown in Fig. 1, the two-colored curves are completely identical. The red curve represents the results taken from OK.Column tool whereas the blue points represents the results taken from ShortCol design tool. ShortCol is a commercial design tool that follows seven different design codes and it calculates the axial force and bending capacity for short reinforced concrete columns of rectangular and circular sections [7]. As shown in the figure above, the red curve is smoother than the blue since it consists of 2000 points. Each point of those 2000 points has a different reduction factor from the others since the reinforcement strain changes along the curve. The 12N24 bars means that there are 12 reinforcement bar of 24 mm in diameter. The above figure follows the Australian design code and the section properties are in SI unit's system.

The interaction chart in Fig. 2 represents a short circular reinforced concrete column with 500 mm in diameter and 50 mm as a cover. The material properties for the same section are shown in the figure below.
The above interaction chart follows the American design code ACI-318 and all the section properties are in SI units system. The circular section has a 12 reinforcement bar of 24 mm in diameter.

The chart shows identical results for both OK.Column and ShortCol tools. However, the difference between them is that the red curve is slightly smoother than the blue curve. The red curve represents the results that comes from OK.Column design tool whereas the blue points represents the results that comes from Short Col design tool.

Another example for circular section interaction chart is shown in Fig. 3.

As shown in the figure above, the two curves are almost identical except for a slight difference when the interaction chart moves from the compression area to the tension area. This difference comes from the reduction factor. ShortCol used a fixed value for both alpha and betta whereas the OK.Column used an equation (shown in the tables above) that calculates alpha and betta whenever $f'_c$ changes according to the Australian and American design codes. The red curve represents the results taken from OK.Column and the blue points represents the results taken from ShortCol.

Fig. 4 shows the interaction charts for the same section but each chart represents different design code. The blue curve represents the American design code ACI-318 and the grey curve represents the Australian design code AS3600.

The two curves are not identical and this is because of the difference in the reduction factors and the alpha and betta variables. From the figure, it can be concluded that the Australian design code tends to use lower
reduction factors so the structure would be safer whereas the American design codes pushes the limit of the resistance factors slightly higher in which it gives higher results than the Australian design code [12].

![Figure 4. Interaction chart for rectangular section, AS3600 & ACI-318](image_url)


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Conflict Detection System in Construction Site Using BIM Technology

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Keywords: Building information modelling (BIM), Conflict, Construction Site, Safety management.

Safety in construction site is a major challenge facing the global building industries. Conflict is one of the most common challenges because construction is a dynamic and complex process that includes a large number of activities and a large number of workers. Despite the improvement in construction safety in many countries, loss of life due to accidents inside the project site still occur. The aim of this study is to enhance the safety in construction site by integrating workspace management with Building Information Modelling (BIM) technology to prevent conflict between labour, project resources spaces and equipment within the construction site. To achieve the aim of this study, several BIM tools have been used including Revit, AutoCAD, and BIM 360. The major contribution of this study is developing the site conflict detection system by using BIM tools for both spatial-temporal conflicts on construction site. This approach will increase and enhance the speed and accuracy of the process of detection the possible spatial conflict.

1. Background

In the last decade, design tools have moved from 2D to 3D modelling [1]. Some software companies, such as Autodesk have produced new design software based on the concept of Building Information Modelling (BIM) [2], which is ‘a digital representation of physical and functional characteristics of a facility’ [3]. Recently, BIM has been one of the most vital development processes in the Architecture, Engineering and Construction (AEC) industry. Several studies have focused on the importance and benefits that should be gained from BIM implementation in the AEC industry [1, 4-7]. Many firms have also been analysing the contribution of BIM to mitigating several challenges facing the construction industry, such as low productivity and the impact of operation costs [8]. In addition, the benefits for different parties of BIM applications have been investigated [9].

There are significant benefits to be gained from adopting BIM applications in the AEC industry. From an owner’s perspective, BIM will decrease financial risks, tighten the project schedule, optimise building performance, increase the reliability of cost estimation, improve project management and enhance facility maintenance. For architects, BIM applications will allow better integration between disciplines, optimise conceptual design and provide for the improved organisation of construction documentation [10]. BIM achieves this by enhancing design simulation and analysis and optimising building design [11]. Although, many studies have researched BIM applications thoroughly, empirical investigation to identify the barriers to BIM has been limited [4]. It is argued that neglecting to study BIM barriers and their effect on the development of the AEC industry has contributed to the slow spread of BIM globally [5]. Many researchers highlighted the significant benefits of BIM adoption in the AEC industry to improve safety management. The empirical evidence from Wan, Platten [12] study suggests that BIM can enable safety systems in construction organisations. Recent research has suggested that BIM can be used for safety checks systems based on existing standards of safety [13-15]. Another direction applied in construction safety was by combining BIM with other technologies such as Radio-Frequency Identification (RFID) to address the blind
spot issues while using multiple cranes on a construction site, [16-19] or combine with Bluetooth low-energy (BLE) based location detection technology [20].

Another benefit of combined BIM with other technologies was highlighted by Bennett and Mahdjoubi [21] study when they used BIM software with cloud technology for maintaining construction health and safety purposes. Another significant use of BIM is for safety training to identify hazards in virtual reality environments [22]. They developed a System for Augmented Virtuality Environment Safety (SAVES) as explained in Chen and Luo [23] studied many aspects that BIM have been applied for in construction industry and one of these aspects was safety. Their model was tested to show the benefits of BIM implementation and its important role in safety, health, environmental protection. Yalcinkaya and Singh [24] studied the essential roles of BIM in AEC industry. They found that the use of BIM applications in safety management is growing rapidly. The role of BIM as a virtual design and construction medium to reduce uncertainty, improve safety, identify problems and analyse potential impacts has been discussed.

Wetzel and Thabet [15] proposed a framework to support safety during the facility management phase through BIM model. The contributions of this study are (1) Identification and classification of FM relevant safety attributes and (2) Develop a data processing and rule-based system (DPRBS) to consolidate and process the classified attributes. Hallowell, Hardison [25] study provide an assessment of the current state of the information technology use to improve construction safety. They present series of studies that provide data to enhance safety management gained from implementation of information technology.

Kang, Youn [26] developed model input to foster collaboration and social interaction, direct manipulation for user system interaction and simulation item creation for real-time network transmission and plan synthesis. The findings from the methodology developed propose further enhancement in the approach including BIM adoption for conveniently obtaining from Product Breakdown Structure (PBS) information, more efficient PBS collection using direct manipulation, and more reliable mechanisms for conflict detection and elimination in the simulation. Ding, Zhou [27] studied the application of BIM in quality, safety and environmental management. For safety management, they found that project site would be safer if BIM applications applied to analyse how different risks combine in different project activities.

The most significant limitation has been found in literature review is that researchers do not study simultaneously and holistically all BIM capabilities when integrated with workspace management. In other words, there is a significant benefit in BIM tools can be useful to four important processes includes workspace generation and allocation, workspace congestion and conflict detection and conflict resolution which represent the essence of workspace management. So, the main aim of this study is to develop a model that can improve the safety in the project site.

2. Conflict Detection

2.1. Workspace Conflict Detection Process

Usually, a physical conflict can occur if the mutual activities are placed at the neighbouring location, therefore, regarding the constructability of each activity, overlapping should be considered, and the use of available resources should be planned efficiently in the planning stage. The bounding box model for the construction equipment can detect any possible conflict on the site (see Fig. 1). The reason is the box surrounding the equipment and location of the project resources both are known. If the activity begins after the start date and before the end date of the neighbouring activity, the conflict will be determined by the overlapped durations between these two activities in the project schedule. Another problem is the
workspace congestion which will occur when the resources of an activity or group of activities are smaller than adequate space. The most important factor which has the significant impact on workspace congestion is supply and demand of resources on project site, therefore, congestion may occur even when there are no spatial and temporal conflicts.

![Figure 1. Conflict area](image)

The conflict in the construction site will occur in two possible situations first if the start date of activity 2 (see Fig. 2) is greater than start date current activity and the start date of activity 2 is less than the end date of current activity. Second, if the end date of the activity 2 is greater than the start date of the current activity and the end date of activity 2 is less than the end date of current activity. Another situation is when if both start and end date for activity 2 between start and end date of the current activity schedule overlap must be checked. Also, the both start and end date of the current activity are between start and end date of activity 2, in this case, schedule overlap must be checked as shown in Fig. 2.

After the workspace overlap is identified, both of activities are considered as a target of checking process because there is a high probability of both schedule and workspace conflict. However, the conflict will be recorded as not occurred if there is no physical conflict between 3D workspace models even schedule overlapping occurred. In this way, the conflicting status of the workspace can be quickly analysed by time-space trade-off according to the schedule overlapping. The first step is to identify any contact point(s) between workspaces inside project site. If such a contact points exist, the corresponding coordinate location should be determined to identify base conflict space which is recognized as the generation of interference between the two or more 3D model workspaces. Then the interference distance between the two workspaces is calculated. The linear distance will be calculated by connecting the virtual lines among the intersection points then by using Revit function to measure 3D distance to extract the biggest of these distances and by doing this the physical adjacency distance between the two workspaces is identified. Physical conflict can determine the adjacency space which is the distance among workspaces and it calculated by measuring the distance between two intersecting surfaces of the two conflict workspaces. Also, the overlapping between activities and equipment workspace should be determined before performing a workspace conflict analysis.
2.2. Workspace Conflict Resolution

Once the workspace conflict verification process is completed or is progressing, the adjacency distances between the workspace models of two overlapping activities are estimated by referencing the geometric properties of the workspaces, assigned to the activities at the same time. The base workspace for the base activity is identified, and then the conflicting workspace, which is a reference workspace, for the following conflicting activity is checked. The conflict types for two workspaces should be determined by recognising the base and conflicting workspaces automatically according to the schedule overlapping results throw Revit software. The distance between two adjacent workspaces is a value derived by calculating the greater distances between surfaces of the two workspaces. The x, y and z coordinates of any specific points on the surfaces are extracted to calculate the shortest distance between two workspaces. The adjacency distance is also computed by substituting the coordinates for the 3D distance calculation in Revit 2017.

The significant role in this research is to BIM software. The Revit, Engineer-to-Order, AutoCAD, Scaffold Counter by Avontus and BIM 360 which have been used in this research.

Figure 2. Conditions of schedule conflicts

Figure 3. Process of Conflict Detection
3. Conclusions

The use of proposed method can be applied in any projects regardless the level of risk or value of the project. A workspace conflict analysis system based on BIM software was developed using this suggested model. The system enables the real-time detection of workspaces’ conflict. If project managers can check workspace conflict between activities by this system, they can reschedule the conflicted activities within float time to minimise the conflict and maximise the constructability for concurrent activities. As a result, the safety on the site could also be improved through the minimization of risk associated with accidents and workspace conflicts. Using the suggested workspace analysis method, a project manager can have an optimal schedule plan in which the workspace conflict is minimised.

This study introduces a method that assists the detection and solving workspace conflicts on a construction site. Workspace generation process was based on bounding box concept that allows to user input for simply adjusted depending on the size of workspace and could be used to represent various types of workspace. The major contribution of this research is to developing the site conflict detection system by using BIM tools for both spatial-temporal conflicts on construction site. This study findings can increase and enhance the speed and accuracy of the process of calculating the possible spatial conflict. The use of proposed method can be applied in any projects regardless the level of risk or value of the project. A workspace conflict analysis system based on BIM software was developed using this suggested model. The system enables the real-time detection of workspaces’ conflict. If project managers can check workspace conflict between activities by this system, they can reschedule the conflicted activities within float time to minimise the conflict and maximise the constructability for concurrent activities. As a result, the safety on the site could also be improved through the minimization of risk associated with accidents and workspace conflicts. Using the suggested workspace analysis method, a project manager can have an optimal schedule plan in which the workspace conflict is minimised.

Limitation and Future Research

The findings of this study have shown that there is a need to identify the location of every equipment and temporary facilities within the construction site. Limitation is that focus of the research was only on the spatial-temporal information. This study was conducted in a virtual site therefore a future research must be conducted to apply this approach to a construction site in a real-world situation for more verifying with a case study for this model and for more on-site knowledge to enhance the accuracy of safety for the construction site.

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Safety Performance of Signalised Intersections and Roundabouts

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Keywords: Crash rate, crash severity, traffic volume, frequency of crashes

The purpose of this paper is to compare safety performance of signalized intersections and roundabouts. A sample of 15 pairs (30 intersections) of these intersection types that have similar characteristics and traffic volumes were selected from different major roads in Metropolitan Melbourne. Crash data at these sites was collected over 5 year period. The comparative analysis was applied to crash rates, frequency, severity and type in addition to a number of situational factors. The letter includes surface condition (wet or dry), number of people involved, number of vehicles involved, and geometric variations. The results of the analysis showed that generally roundabouts are associated with lower crash frequencies and rates than signalized intersections in addition to being associated with crashes of lower severity.

1. Introduction

Over the past few years, roundabouts started to be considered safer than other types of intersections due to the fact that these roundabouts have the ability to reduce speed of the approaching cars and decrease the number of conflicts between road users. According to [4], the number of crashes that occur at signalized and give way intersections can be reduced by reducing the number of conflict points between the two geometric configurations. Also, [4] has defined the conflict point as a point where a vehicle can cross another vehicle’s travelling path. Further, number of conflict points can be reduced from 32 conflict points to only 8 conflict points by using roundabouts instead of cross-intersections, which will absolutely reduce the possibility of having a crash. Nevertheless, roundabouts have shown that other types of road users (e.g. cyclist, pedestrian) are more commonly involved in road crashes. [3] Stated that roundabouts with cyclist close to the roadway are performing worse than signalized intersection and off-road cyclist path. The main aim of this paper is to conduct an analysis and a comparison that can give clear responses about the safety performance of signalized intersection and roundabouts. The evaluation of these two intersections will also be based on many other factors.

2. Literature review

A study has been conducted [2] in Australia about the safety performance of intersections. The aim of the study was to improve intersection’s safety performance by applying specific modifications to existing infrastructure, using multifaceted and multidisciplinary approach. The crash data that have been used in analytical part of this report was collected in Victoria between 2000 and 2005. [2] Stated that the statistical analysis of the data showed that 45% of the individuals that were involved in crashes were either killed or suffered from serious injuries. Also, the authors stated that 48% of the injuries occurred at cross-intersections. Further, they stated that 52% of the fatal crashes occurred in 60 km/h speed zone.
Their study also showed that the least number of fatal crashes occurred in Sunday, while the greatest number occurred on the Friday which was 40% higher than Sunday crashes. Further, the highest number of fatality crashes occurred during peak-hours in the afternoon between 4pm and 7pm.

According to [6], over recent years there has been a trend to use roundabout for intersection control. Therefore, several sites were selected in Victoria (Plenty Road, McDonalds Road, Gorge Road and South Morang intersections), to determine the safety performance of these roundabouts. The observations of this study were that the deflection path of the approach and the location or sizes of the central island are the most important geometric features that effect on the safety performance of roundabout. The study showed that inadequate deflection path or location and size of the central island led to high travel speed which increased crashes and crash severity.

The statistics showed that the crash frequency rate for the selected sites over period of 5 years from 2000 to 2004 is 1 crash per year. Also, it showed that the severity crashes represent only 18% of the crashes over 10 years period. In addition, the study estimated that the crash frequency is 1.7 crashes/year/107 entering vehicles.

Nearly half of vehicle crashes in United States during 1998 have occurred at intersections, with an approximately 8600 fatal crashes and 937000 non-fatal crashes. A study has been done by [5] on 24 intersections using the Bayes approach to estimate the potential reductions in vehicle crashes by converting these signalized intersections to modern roundabouts, using simulation. Results of showed that using roundabout instead of a signalized intersection can produce substantial reduction in vehicle crashes. According to [5], using roundabout as an alternative to signalized intersection can reduce injury crashes by 76% in general and reduce sever injuries by 38%. Also, it can reduce fatal crashes by 90%.

[5] States that crashes reduction that resulted from the conversions can be attributed primary to two factors:

- Reduced vehicles speeds
- Eliminating specific types of vehicle conflicts that occur at intersections

These conflicts include right-angle conflicts at stop signs and signals, left turn against opposite conflicts and front to near conflicts. These conflicts consider as 70% of police reported crashes in United States intersection crashes. Therefore, according to [5], installation of roundabouts should be considered as an effective safety action.

[4] Reported a study on five single-lane roundabouts in Florida and Maryland that were converted from signalized intersections. The study included a comparison of the selected sites in terms of safety performance and operation performance before and after roundabouts installation. [4] Stated that roundabouts reduce the delays and crashes due to unique geometric characteristics that promote a reduction in fatal crashes. For instance, a signalized intersection has 32 conflict points, while a roundabout has only 8 conflict points. This reduction in conflict points can help the roundabout to reduce crashes. Also, the types of crashes that occur at roundabouts do not include left turn head and right-angle crashes. Roundabout’s unique geometric design also helps to reduce fatal and injury crash.
Stated that there was another study that has been conducted in Victoria, Australia, on 73 sites before and after roundabouts installation. The study showed that there was a reduction by 74% in casually crashes. Also, there was a reduction by 32% in property damage crashes and a 68% reduction in pedestrian crashes per year for the installed roundabouts [1].

3. Methodology

3.1 Site selection

The safety performance of roundabouts and signalized intersection has been compared according to the data that has been collected from 30 different sites. These sites should include an equal number of both signalized intersections and roundabouts i.e. 15 pairs where every pair is located on the same road to ensure similar traffic volumes, environmental and operational factors. The criteria used in selecting the sites are summarized below:

- Traffic volume i.e. the number of cars that pass a specific part of the road in a specific period of time.
- Each pair of signalized and roundabout intersections should have similar traffic volumes
- Turning movements should be similar for both types of intersections
- Locations i.e. each pair of signalized and roundabout intersections should be located in a same road.
- Geometric design i.e. size of intersection in terms of number of legs, cyclist facilities, etc.

Finding exact location for each roundabout and signalized intersection has been done by the authors using the Melway and Google maps. Melway can provide an accurate coordinate for the located sites. For the purpose of this study, all the crashes data are in period between 2009 and 2013.

3.2 Data collection and preparation

Crash data was collected from crashstat provided by VICROADS and included information on crash types, severity, location, speed, surface condition, types of vehicles and users involved and their numbers, time of day, gender and age of users involved, time of day (day/night), and surface condition (wet/dry). As for traffic volume data, it was extracted from excel file provided by VICROADS for the relevant years. However, not all this data is going to be discussed in this paper but only crash frequency and rates, locations, user and vehicles involved, and surface condition (wet/dry).

The main purpose of the research has been achieved through analysing several sites that have witnessed crashes under comparable conditions. These sites were collected and grouped according to their traffic volume and geometric design. During the sites analysis we examined injury severity at different levels, number of vehicles and people involved in a crash and rates of crashes for each intersection. Throughout the analysis, most of the factors showed higher numbers of crashes occurred at signalized intersections compared to roundabouts. However, since not all measures and factors are likely to be influenced to the same extent, we will provide some considerations below and whether they can affect our conclusion or not.
4. Results

The first safety performance measure is crash severity. (Fig. 1) shows the number of serious injuries crashes, fatal crashes and other injuries for the 30 selected sites. From (Fig. 1) it can be observed that the number of fatal crashes is zero for roundabouts and two for signalized intersections.

While the numbers of serious injuries are 11 injuries for roundabouts and 36 injuries for signalized intersections.

In addition, other injuries were 29 injuries for roundabouts and 57 injuries for signalized intersections over the same period.

Table 1 summarizes the crash rates for 15 pairs (30 intersections) that have been chosen for 5 years period from 2009 to 2013. It also shows the average crash rates which were 0.19 for roundabout and 0.64 for signalized intersection.

It is clear from the table that the traffic volume of each pair is close enough to compare between the two types of intersections. Also, the geometric design is same since both of compared roundabout and signalized intersections have the same number of legs as shown in Table1.

Moving to one of the effective factors, which is the number of involved people, the analysis showed that the number of crashes occurred at roundabouts is 6, 26 and 9 for one person, two persons and three persons involved respectively. While, the number of crashes that occurred at signalized intersections is 2, 45 and 47 for one person, two persons and three persons involved respectively as shown in (Fig. 2).
For the number of vehicles involved, the analyses showed the number of crashes is 03 at roundabouts and 65 at signalized intersections for two vehicles involved in the accidents, while for one vehicle and three vehicles the number of crashes is much lesser at both roundabout and signalized intersection (Fig. 3).

5. Discussion

5.1 Crash rates

In the examined data, the crash rates have increased at signalized intersections when traffic volume was low at those intersections. This means that signalized intersections are less efficient with low traffic volume. On the other hand, roundabouts that have been examined showed a more consistent performance during high and low traffic volume, since the crash rates were fluctuating between 0.00 and 0.56 unlike signalized intersections which were fluctuating between 0.00 and 3.90 crashes per million passing car. Also, the study showed that the highest crash rate for signalized intersections occurred at Kororoit Creek road which were 3.90, in contrast, the rate was 0.28 for the roundabout that is located in the same road. Generally, the average crash rate of all the signalized intersections was 0.64, while it was 0.19 for all roundabouts. This can give us a clear indication that roundabouts are performing better at low and moderate traffic volume especially when we know that 35% of all roundabouts have not witnessed any crashes during the examined period.

5.2 Crash severity

Regardless of the severity levels in the observed data, injuries and fatal crashes were presented in our data. Different way of analyzing the data showed that the percentages of severe crashes were highly increased for signalized intersections compared to roundabouts. Furthermore, no fatal crashes were recorded among all the roundabouts that have been considered. However, signalized intersections have showed two fatal crashes during the examined period. Regardless of the type of the road user and any other affecting factors, the numbers of crashes that result in serious injuries were three times higher for
signalized intersections compared to roundabouts. Therefore, it is clear that roundabouts are performing better that signalized in terms of the fatality, involvement and injuries.

5.3 Other contributing factors
This paper focused only on some of the affecting factors that could increase the crash severity. As mentioned in the results that the involvement of more than three people was significantly higher for signalized intersection compared to roundabout as shown in (Fig. 2). This can increase the severity of crash due to fact that more people can killed or injured at signalized intersections. The analysis has also shown a considerable difference in the number of involved vehicles, which also could lead to increase the crash severity. The last affecting factor has been considered is the surface condition. The examination of the data showed that 10.8% of the accidents that occurred during a wet condition surface took place at roundabouts and 89.2% took place at signalized intersections. From this result, it is clear that roundabouts are performing safer than signalized intersections in terms of number of vehicles and people involved and the surface condition.

6. Conclusion
This paper described a comparison of different types of intersection based on a data provided by VICROADS. The differences in the geometric design of signalized intersection and roundabout have been considered as significant contributor to their safety performance. Design of signalized intersection is considered more complex compared to other types of intersections, such as, standard roundabouts. This complexity is because of the varieties of traffic management parameters that control the intersection traffic lights. However, the deflection path that is provided in the roundabout could reduce the severity of crashes and the number of conflict points is less for roundabout compared to cross-intersections.

From the analysis of the provided data, three main measures (crash rate, crash severity, crash frequency) were obtained by analyzing different scenarios. The crash rates showed that signalized intersections are less efficient with low traffic volume. On the other hand, roundabouts that have been examined showed a more stable performance. Also, the numbers of crashes that result in serious injuries were three times higher for signalized intersections compared to roundabouts, which gives us a clear indication that roundabouts are performing better. Further, roundabouts performance in terms the contributing factors to the crash severity was way much better compared to signalized intersection and as indicated in the discussion.

Acknowledgement
Support from Dr. Rayya Hassan, Dr. Raidh Al Mahadi, Swinburne University, Baghdad University and VICROADS are gratefully acknowledged but the views expressed by the authors do not necessarily reflect those of the individuals and organizations. The authors also would like to thank their families and friends for their support.
References


Hybrid simulation system and its applications in collapse assessment of limited-ductility bridges

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Keywords: experimental methods, hybrid simulation, bridge piers, limited ductility

Collapse assessment of structures requires the prediction of a structure’s response from the initial linear-elastic behavior to collapse and thus poses significant challenges. Seismic evaluation of structural systems has traditionally been explored using either experimental methods or analytical models. However, the development and use of advanced cyber-physical systems has paved the way for structural and earthquake engineers and provided the opportunity to enhance the existing experimental methods of examining the performance of novel smart structures in a suitable and cost-effective manner. Hybrid simulation is a cyber-physical testing technique that overcomes many of the limitations of shaking tables while using similar equipment used for quasi-static testing, making it a versatile and cost-effective experimental method to evaluate the dynamic performance of large-scale structures. Hybrid simulation system including Multi-Axis Substructure Testing (MAST) system in the Smart Structures Laboratory (SSL) at Swinburne University of Technology provides a powerful tool for investigating the dynamic effects of earthquakes, hurricanes, and other extreme loading events through local or distributed hybrid simulation of large or full scale structural components. The unique and versatile capabilities of the hybrid simulation system at Swinburne are discussed in this paper, which will greatly expand the capabilities of large-scale experimental testing.

1. Introduction

Natural hazards, such as earthquakes and strong winds are the largest potential source of casualties for inhabited areas. Damages to structures cause not only loss of human lives and disruption of lifelines, but also long-term impact on the local, regional, and sometimes national and international economies.

One of the main goals of structural and earthquake engineering is to improve the understanding of earthquakes and their effects on the structural systems and non-structural components. Accordingly, in order to develop new smart materials, new devices and technologies and new smart structural systems for extreme dynamic load-resistant structures, the priorities lie on gaining an understanding of the behaviour of various classes of structures under different dynamic load types from elastic range through failure and developing collapse mechanisms. However, reliable assessment and prediction of nonlinear structural behaviour and their failure mechanism has proven to be an extremely difficult task.

Nowadays, dynamic analysis of complex structures can be efficiently computed utilizing different available software. However, earthquake engineers still rely on experimental testing methods since the seismic response of structural systems is extremely complex. This is, firstly, due to the uncertainty associated with the occurrence of the earthquake that does not allow the exact evaluation of the seismic demands on the structures and secondly, the needs for the knowledge of nonlinear dynamic response of the materials and elements over the full range of the seismic response, from linear elastic range to levels approaching
collapse. Therefore, laboratory testing still has significant importance for the research community for verification and further development of numerical models and their calibrations.

Currently, there are three types of experimental testing procedures used to evaluate structural behaviour subjected to dynamic loadings: shake table testing, quasi-static testing, and hybrid simulation [1]. In shake table testing, realistic test conditions can be produced to evaluate the dynamic behaviour of civil structures. In this method, some critical issues such as collapse mechanisms, component failures, acceleration amplifications, residual displacements and post-earthquake capacities can be investigated. Nevertheless, very few shake tables in the world are capable of testing full-scale large civil structures. Therefore, shake table testing is excessively expensive, limited to the load-bearing capacity of the testing platform and the interaction between the specimen and the shake table.

Quasi-static testing is another technique used to evaluate the dynamic performance of civil structures. Commonly, this technique is applied to study the hysteretic and cyclic behaviour of structural components subjected to seismic loading. Even though quasi-static testing can be implemented on large civil structures, it has two major drawbacks. Firstly, it requires a pre-defined displacement history, which is generally inadequate for resembling the structural behaviour as the load distribution continuously changes during an actual seismic event. Secondly, the effect of the specimen’s nonlinear behaviour on the overall response cannot be studied since there is no interaction between the specimen response and the pre-determined loading sequence.

Evolved from pseudo-dynamic testing [2], hybrid simulation is a versatile and economically viable experimental technique to evaluate the dynamic performance of large civil structures [3]. According to a report developed by the US earthquake engineering community in 2010, hybrid simulation capabilities are a major emphasis of the next generation of earthquake engineering research [4].

Hybrid testing provides an attractive alternative for safe and economical dynamic testing of structural systems over the full range of the seismic response, from linear-elastic range to levels approaching collapse. It facilitates the study of the structures by experimentally evaluating only the critical portion of the structure while the rest of the structure, inertia and damping forces, gravity and dynamic loading and second order effects are modelled numerically in the computer. During the simulation, the physical portion of the overall hybrid model is tested in one or more laboratories using computer-controlled actuators, while the numerical portion is analysed on one or more computers [5]. Since dynamic aspects of the simulation are handled numerically, such tests can be viewed as an advanced form of quasi-static tests, where the loading history is determined as the simulation progress for the structure of interest subjected to a specific ground motion. The governing equation of the motion is solved similar to pure numerical simulations using a time-stepping integration. The displacement demands are then applied to the physical specimen and the resisting forces are measured and fed back to the computation solver to calculate the displacements corresponding to the next time step.

2. State-of-the-Art System for Hybrid Simulation at Swinburne

The hybrid simulation system in smart structures lab (SSL) at Swinburne consists of several components including software and hardware that allow for hybrid testing in various configurations. Currently, the experimental hybrid procedures include scaled-time hybrid testing (pseudo-dynamic) with substructuring but can be extended to real-time hybrid simulation and effective force testing.
An advanced hardware configuration has been set up to ensure a strong coupling and a very high-speed data communication between the servo-controllers and the main computer solving the equation of motion. Hybrid simulation frameworks include:

1. Multi-axis substructure testing (MAST) system for three-dimensional large-scale structural systems and components (suitable for users).
2. 1MN universal testing machine that is suitable for SDOF tests (suitable for developers).
3. Generic actuator configuration system for substructure hybrid tests (suitable for both developers and users).

The MAST system in the Smart Structures Laboratory at Swinburne (Fig.1) advances the current state of technology by allowing the experimental simulation of complex boundary effects through its multi-axial capabilities. The unique and versatile capabilities of the MAST system will greatly expand the experimental testing of large-scale structural components such as beam-column frame systems, walls, bridge piers, etc. Using MAST system, the developments of new materials and structural components, and the effectiveness of new retrofit strategies for seismically damaged structural elements can be reliably evaluated through three-dimensional (3D) large-scale hybrid simulation, which provides significant insight into the effects of extreme loading events on civil structures. The capabilities and specification of MAST system is discussed next.

2.1. MAST System in Smart Structures Laboratory at Swinburne

Multi-Axis Substructure Testing system in the Smart Structures Laboratory at Swinburne provides a powerful tool for investigating the effects of earthquakes, hurricanes, and other extreme loading events on large structural components using hybrid simulation testing. A sophisticated 6-DOF control system is used utilizing eight high-capacity hydraulic actuators that enable application of complex multi-directional deformation or loading schemes to structural components. The MAST system has the capability to accommodate test specimens with up to 3m×3m in-plan and approximately 3m height.

2.1.1. MAST Laboratory

The MAST system is located in the Smart Structures Laboratory at Swinburne university of Technology. The laboratory is a major 3D testing facility developed for large-scale testing of civil, mechanical, aerospace and mining engineering components and systems and the only one of its type available in Australia. The 1.0m thick strong floor measures 20m×8m in plan with two 5m tall reaction walls meeting at one corner. The 3D strong cell contains a grid of tie down points 0.5m apart to secure the test specimens in place, in addition to a suite of hydraulic actuators and universal testing machines varying in capacity from 10tonnes to 500tonnes. The laboratory is serviced by adjacent workshops and a hydraulic pump system located in the basement. The facility is housed in a large architecturally designed test hall about 8m tall.
2.1.2. MAST Actuators

Two sets of actuator pairs with strokes of ±230mm provide lateral loads up to ±500 kN in the orthogonal directions. These actuator pairs are secured to the L-shaped strong-wall. Four ±1 MN vertical actuators, capable of applying a total force of ±4 MN with strokes of ±230 mm, connect the crosshead and the strong floor. Auxiliary actuators are also available to be used for additional loading configurations on the specimen. The actuator system specifications are presented in Table 1.

**Table 1. MAST system specifications**

<table>
<thead>
<tr>
<th>MAST Actuators Capacity</th>
<th>Vertical</th>
<th>Horizontal</th>
<th>Auxiliary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>MTS 244.51</td>
<td>MTS 244.41</td>
<td>2 (MN)</td>
</tr>
<tr>
<td>Quantity</td>
<td>4 (Z₁, Z₂, Z₃, Z₄)</td>
<td>4 (X₁, X₂, Y₃, Y₄)</td>
<td>(Qty. 1)</td>
</tr>
<tr>
<td>Force Stall Capacity</td>
<td>± 1,000 (kN)</td>
<td>± 500 (kN)</td>
<td>(Qty. 3)</td>
</tr>
<tr>
<td>Static</td>
<td>± 250 (mm)</td>
<td>± 250 (mm)</td>
<td>25 (kN)</td>
</tr>
<tr>
<td>Servo-valve flow</td>
<td>114 (lpm)</td>
<td>57 (lpm)</td>
<td>10 (kN)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAST DOFs Capacity (non-concurrent)</th>
<th>Load</th>
<th>Deformation</th>
<th>Specimen Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>DOF</td>
<td>1 (MN)</td>
<td>± 250 (mm)</td>
<td>3.00 (m)</td>
</tr>
<tr>
<td>X (Lateral)</td>
<td>1 (MN)</td>
<td>± 250 (mm)</td>
<td>3.00 (m)</td>
</tr>
<tr>
<td>Y (Longitudinal)</td>
<td>4 (MN)</td>
<td>± 250 (mm)</td>
<td>3.25 (m)</td>
</tr>
<tr>
<td>Z (Axial/Vertical)</td>
<td>4.5 (MN.m)</td>
<td>± 7 (degree)</td>
<td></td>
</tr>
<tr>
<td>Rx (Bending/Roll)</td>
<td>4.5 (MN.m)</td>
<td>± 7 (degree)</td>
<td></td>
</tr>
<tr>
<td>Ry (Bending/Pitch)</td>
<td>3.5 (MN.m)</td>
<td>± 7 (degree)</td>
<td></td>
</tr>
<tr>
<td>Rz (Torsion/Yaw)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2. Hybrid Simulation Architecture

The hybrid simulation control system at Swinburne uses xPC-Target and consists of a three-loop architecture, which is depicted in (Fig. 2). The innermost servo-control loop contains the MTS FlexTest controller that sends displacement/force commands to the actuators while reading back measured displacements/forces. The displacements are measured from both the actuator LVDTs and high-precision...
string potentiometers. The middle loop runs the Predictor-Corrector actuator command generator on the xPC-Target [6] real-time digital signal processor (DSP) and delivers the displacement/force commands to the FlexTest controller in real-time through the shared memory SCRAMNet [7]. Finally, the outer integrator loop runs on the xPC-Host and includes OpenSees [8], MATLAB [6] and OpenFresco [5] that can communicate with the xPC-Target through TCP/IP network.

![Figure 2. Hybrid Simulation Architecture at Swinburne](image)

3. Test structure

The research shows that limited ductility bridges are often at the verge of collapse during earthquakes for two reasons. First, they might not have sufficient capacity to withstand design loads. Second, past experiences have shown the intensity of the earthquake could reach twice of the design loads. Hence, it is important to quantify the reserve capacity of the bridges to have a better understanding about the likelihood of collapse. The reserve capacity is acquired through multi-axis testing of a column with similar material properties and load pattern from a previous study [9]. After capturing the full response of the column from elastic range to collapse level, the model can then be calibrated and used to perform the incremental dynamic analysis (IDA). IDA consists of applying a suitable suite of ground motions a nonlinear model of the structure to study the expected structural response and damage outcomes. IDA is used to develop fragility curves about the uncertainties of the ground motions to quantify the probability of collapse.

OpenSees is used for modelling the structure [8]. The frame’s elements are modelled using BeamWithHinges element type to consider both bending and axial loading. The nonlinear behaviour is demonstrated by using a distributed-plasticity concept where the plastic behaviour takes place in a finite length near the ends of the beam-column element. The lumped plasticity model followed peak-ordinated hysteresis response based on the Modified Ibarra-Medina-Krawinkler (IMK) deterioration model [10] for the flexural behaviour (Fig. 3a).
The moment-curvature properties of the nonlinear sections in the beam and column elements are calibrated to the modelling parameters predicted from a series of empirical equations. These empirical equations relate column design characteristics to IMK modelling parameters [11] (Fig. 3b).

![Graph](image.png)

**Figure 3.** Simulation of nonlinear hinges using IMK peak-oriented model.

4. Hybrid simulation

Prior to conducting the hybrid simulations with the physical subassembly in the laboratory, a series of coupled numerical simulations must be conducted to evaluate the integration scheme parameters for the actual experiments. These numerical studies serve as verification of the substructuring techniques by demonstrating that a partitioned frame model using this approach provides results similar to a full frame model. In the coupling simulation method of two substructures presented by Schellenberg [12], the numerical substructure takes the role of the master simulation and consists of the numerical subassembly. For these numerical studies, a separate numerical model is provided to represent the experimental substructure and acts as the slave. Both master and slave subassemblies are modelled in OpenSEES in these simulations. The equation of motion is solved for the complete system with the slave acting as an ‘element’ of the master simulation.

During the analysis, the master imposes the displacements on the boundary of the slave subassembly and the slave program returns corresponding forces back to the master program. A generic super-element is connected to the master model and utilized to provide the interactions with the slave model. In addition, adapter elements are added to the slave model to provide the interface with the master model, essentially acting as the actuators to control DOFs in the slave model. OpenFresco is used to connect the master and slave programs through their generic and adapter element interfaces. Later in the actual hybrid simulations, the slave is simply replaced with the experimental substructure. In this study the slave model is the left experimental bridge piers (Fig. 4a).

The procedure involves subjecting the structure to the bi-directional ground motion of Loma Prieta 1989 with sequentially increasing intensities. Then, from the incremental dynamic analysis, two levels of ground motion will be selected to cover the structure’s response from linear-elastic range through collapse [13]. Preliminary numerical studies conducted for the accuracy and stability of integration scheme in hybrid
The simulation of a RC-OMRF structure showed identical results comparing the full and hybrid coupled-numerical model, while keeping the unbalance forces below stability margin.

To illustrate this process for the various types of substructures in hybrid simulation, an example is presented for a bridge structure with two piers (Fig. 4a). Utilizing the hybrid simulation technique, one of the bridge piers can be constructed and physically tested in the lab and the remaining parts of the bridge, mass, viscous and friction damping, gravity and dynamic loads and also the second order effects can reliably be modelled in the computer (Fig. 4b).

**Figure 4.** Hybrid simulation technique.

### 5. Conclusion

Large-scale testing of structural components can deliver significant benefits to structural and earthquake engineers. The behaviour of structural elements can be studied by replicating extreme loading conditions that currently cannot be produced by other means. The Multi-Axis Substructure Testing system at Swinburne enables the evaluation of existing systems, retrofitted systems, and new systems and materials to develop durable and economical structural systems capable of resisting seismic, wind, and other types of loading. Key features of the MAST system include but are not limited to: 1) It allows to control 6-DOF (vertical, lateral, longitudinal, pitch, roll and yaw) utilizing a rigid cruciform and therefore reliably simulates the complex boundary effects. 2) It accommodates the testing specimen up to 3meters cubed that is suitable for large-scale substructures. A framework for collapse simulation of structures through hybrid simulation is presented. In this framework the 6-DOF interaction between analytical and experimental subassemblies are controlled during the simulation.
References


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A comparison study between basalt and granite crushed rocks under repeated traffic loads

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Keywords: flexible pavement; unbound granular materials; igneous rocks; repeated load triaxial test; permanent deformation

Class 2 crushed rock is used as an unbound granular pavement base material in Australia. Commonly, there are several types of Class 2 rocks attributed to their origins. Roads Corporation of Victoria (VicRoads) specification assigns several requirements including Los Angeles Value to characterise the strength of the base layer’s crushed rock. As the base materials are exposed to traffic loads, it is more rational if it is characterised under similar loading configurations. Two types of Class 2 rocks are used in this study-crushed basalt and granite rocks. A repeated load triaxial test (RLTT) was conducted to assess the permeant deformation (PD) properties for both types of crushed rocks. The results from the RLTT test show that Class 2 crushed basalt rock has PD varied from 2.0% to 4.3% while PD of Class 2 crushed granite rock varied from 1.0% to 2.3% meaning that granite rocks are comparatively superior to their basalt counterparts.

1. Introduction

Igneous rocks can be divided into four groups, according to the mineral type and the formation process. These groups include plutonic, subvolcanic volcanic rocks and other igneous rocks, which are all formed at different depths and have different grain sizes, mineral and chemical compositions \cite{1}. Some areas of Victoria are geologically much younger as a result of volcanic activity which last erupted a few thousand years ago \cite{2}. The absence of natural aggregate imposes the need of alternatives such as processed crushed rock \cite{3}. Typically, crushed rock manufacturing begins with blasting massive rocks and is followed by several crushing stages. Crushed rock is composed of rock fragments produced by the crushing, scalping and screening of row rocks. Crushing process encompass some modification to match the demand for each type of gradation \cite{4}. Generally, crushed rock is one of the unbound granular materials (UGMs) that used as base and subbase materials in the flexible pavement.

The main purpose of the crushed rock in the base and subbase layers is to withstand the applied traffic loads and environmental conditions and to transmit them to the subgrade to avoid any failure in the surface layer. In addition, increasing in pavement service life, improving in construction quality, and reduction in asphalt concrete thickness could be achieved by a proper selection of the crushed rock of the base layer \cite{5}.

Crushed rock reflects both permanent (plastic) strain and recoverable (resilient) strain. The permanent deformation (PD) is caused by traffic loads and the environmental conditions \cite{6}. The total strain is the summation of both types as mentioned in Eq. (1).

\[
\varepsilon_t = \varepsilon_r + \varepsilon_p
\] (1)
where,

\[ \varepsilon_t = \text{total strain}, \]
\[ \varepsilon_r = \text{resilient strain}, \]
\[ \varepsilon_p = \text{plastic (permanent) strain}. \]

In the Mechanistic-Empirical pavement design methods, pavement distress is an important consideration. The identification of the type of distresses is important as it helps to find the origin of that distress [7].

Rutting is a PD distress. A rut is the longitudinal surface depression on the surface of the under wheel path [8]. Structural rutting in the surface layer usually results from failure in the underneath layers such as base, subbase or subgrade. Rut can be easily observed on the pavement surface when rain water fills the pavement surface depression. Structural failures and a potential for hydroplaning can be caused by rutting. Rutting is measured as a surface area or as a depth [7]. As shown in Figure 3 the compressive PD at the top of layers causes pavement rutting. It is important to determine the PD of the materials of each layer in order to predict the accumulated rut depth [7].

Class 2 crushed rock is a high-quality pavement base material as recommend by Roads Corporation of Victoria (VicRoads). Several requirements are assigned for Class 2 (e.g. liquid limit, plasticity index, flakiness Index and Los Angeles Value). The deformation under repeated loads is the better approach to evaluate UGMs performance (e.g crushed rock) [9].

As the base materials are exposed to dynamic traffic loads, it is more rational if the crushed rocks are characterised according to the mechanical behaviour in addition to the physical properties.

![Figure 3. Rut formation due to high plastic compressive strain](image_url)

This study focuses on PD of Class 2 crushed rocks under repeated dynamic loads. The main objective of this research is to compare the deformation response of Class 2 basalt and granite crushed rocks as an approach to characterise Class 2 according to their mechanical behaviour.
2. Experimental Investigation

2.1. Materials tested

Class 2 crushed rock is used as pavement base material in Victoria, Australia, as recommended by Roads Corporation of Victoria (VicRoads). Two different sources of Victorian igneous rocks used in this study—basalt and granite rocks. Crushed basalt Class 2 rock was collected from Mountain View quarry in Point Wilson and crushed granite Class 2 rock was resourced from Hanson quarry located in Lysterfield. Both quarries located in Victoria, Australia.

The optimum moisture content (OMC) and the maximum dry density (MDD) for Class 2 basalt and granite crushed rocks are calculated in accordance with Australian Standard [10], and tabulated in Table 1.

<table>
<thead>
<tr>
<th>Class 2 type</th>
<th>OMC (%)</th>
<th>MDD (t/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed basalt</td>
<td>7.7</td>
<td>2.32</td>
</tr>
<tr>
<td>Crushed granite</td>
<td>6.0</td>
<td>2.30</td>
</tr>
</tbody>
</table>

Table 1. Modified compaction results of the investigated Class 2

Gradation curve for both Class 2 are between the upper and lower limits of VicRoads’ specification as presented in Figure 2.

![Figure 4. Gradation chart for tested materials](image)

2.2. Specimen fabrication

Dynamic modified compaction is carried out to produce cylindrical samples of 100 mm diameter and 200 mm height. Class 2 for each specimen is mixed with the corresponding OMC and each layer is compacted by 25 blows to bring the sample to the corresponding MDD (Table 1).

![Figure 4. Gradation chart for tested materials](image)

After moulding, a rubber membrane is placed around the specimen which helps to prevent the radial water filtration through the specimen. O rings were utilised to restrain the top and the bottom of the membrane. The final specimen is placed over the triaxial pedestal as shown in Figure 5.
2.3. Testing equipment
The effect on the PD response of Class 2 crushed rocks is assessed by using repeated load triaxial test (RLTT) which has been acknowledged as the most reliable simulation to the field traffic loading configurations [12]. The whole unit consists of load frame, actuator motor, triaxial cell, digital control system, pneumatic controller, external displacement transducer and computer control. Air and water are used as a confining media. Triaxial cell pressure is controlled via pneumatic controller which control air pressure.

2.4. Permanent deformation testing procedure
Three stress stages are imported into RLTT. Each PD involving of 10,000 cycles. For base materials, the repeated deviator stress and the static confining stress magnitudes are stated in Table 2 as per Austroads guide [11].

<table>
<thead>
<tr>
<th>Stages</th>
<th>$\sigma_3$ (kPa)</th>
<th>$\sigma_d$ (kPa)</th>
<th>Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>350</td>
<td>10000</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>450</td>
<td>10000</td>
</tr>
<tr>
<td>3</td>
<td>50</td>
<td>500</td>
<td>10000</td>
</tr>
</tbody>
</table>

Table 2. Stress sequences for permanent deformation (base materials)

3. Data analysis and discussion
The comparison of PD test obtained in the similar conditions for the two tested Class 2 are presented in Figure 6. As listed in Table 2, each stage has different deviator stress and constant confining pressure of 50 kPa. The PD of Class 2 crushed basalt and granite rocks increased with deviator stress and number of cycles as concluded by Werkmeister [13]. The results from the RLTT test shows that Class 2 crushed granite rock has a smaller PD than Class 2 crushed basalt rock. Specifically, Class 2 crushed basalt rock has PD
varied from 2.0% to 4.3% while PD of Class 2 crushed granite rock varied from 1.0% to 2.3%. The reason behind the high PD resistance of Class 2 crushed granite in compared to Class 2 crushed basalt rock could be the mineralogy (Silica content, SiO2) and the cooling rate during the formation process. Since basalt is about 53% SiO2, whereas granite is 73%. The former interpretation is conceivable since the stresses transmitted through the aggregate particles (point contact).

![Figure 6. Permanent deformation of basalt and granite Class 2](image)

3. Conclusion

The results from the RLTT test showed that Class 2 crushed granite rock has a smaller PD than Class 2 crushed basalt rock. The PD of Class 2 crushed basalt and granite rocks has increased when deviator stress and number of cycles increased. Mineralogy and cooling rate (during the formation process of the raw rock) could be the reason behind the variation in PD’s resistance. Since the two types of Class 2 crushed rocks were tested corresponding to OMC and MDD, further tests need to be conducted by varying these values.

Acknowledgement

This is ongoing PhD study under HCED Iraqi Scholarship. We would like to acknowledge the Higher Committee of Education Development (HCED) in Iraq for the scholarship.

References


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Studying the mitigation of the Urban Heat Island (UHI) by a small park in Melbourne city

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Keywords: UHI, Cooling Effect, Urban Parks

The Urban Heat Island (UHI) can be described as a city core always much warmer than the surrounding areas. The difference can reach up to 8°C. This study selected a small park (Argyle Square) in Melbourne to investigate how far can feel the difference in air temperature from park boundaries towards the city centre Park Cool Distance (PCD), and what the magnitude of temperature, which decreased Park Cool Island PCI in the long term. This study used hand-held weather meter device to measure air temperature at 1.5 m above the ground, across 8 transect points extending to 915 m. Study period extended one year to collect data, which repeated 6 times a day over four seasons (one month per season). Field data have been corrected by the nearest ground weather station. Study results presents that the park has a notably cooling effect for a distance Park Cool Distance (PCD) 310 m-915 m from its boundaries, and Park cooling magnitude (PCI) 0°C–7.3°C. Conclusion is small urban park can combat urban heat respective to both of the PCI magnitude and PCD.

1. Introduction

In recent years, one of the impacts that has appeared clearly in many places around the world, particularly in heavily urbanized areas, is known as the Urban Heat Island (UHI). UHI described as air or surface temperature in cities much warmer than rural or adjacent suburb areas [1–5]. UHIs increased yearly with increasing temperature in the urban cities. For example, in terms of air temperature, the UHI magnitude in Shanghai increased with a rate 0.2°C over each 10 years ago [6]. Annual UHI magnitudes varied between 0.31°C–4.9°C in different cities such as Paris, Adelaide, Orlando, Melbourne, Shenzhen, Moscow, New York, Seoul, Granada, Beijing and Alaska [7–17]. UHIs can be noticed in large, medium and small cities [18–20], with different spatial and temporal resolutions [21–22]. The intensity of UHIs varies between seasons. For example, UHIs magnitude in summer higher than the other seasons [23], while other researchers showed that UHIs magnitude in winter higher than the other seasons [24]. UHIs impacts on cities by six aspects: social and environmental issues, human health, thermal plume, air quality, natural sources and plant diversity.

There are three ways have been mitigated UHIs by using: reflective materials, water bodies and vegetation. The most effective ways is using vegetation, there are four formations of vegetation (green roofs and walls, street trees and urban parks). Urban parks are considered the most effective way to decrease temperature around it [25]. Urban parks have effected with different sizes. Besides this, urban parks effects at different measuring time in day and night.

In this study, we selected a small urban park close to Melbourne city to show the effect of the park on both cooling magnitude and cooling distance during a whole year.
2. Study area and Methodology

Argyle Square covers an area of 1.5 ha and is located north of Melbourne in Carlton (37°80′28″S 144°96′62″E) (Fig. 1). Vegetation covers around 70% of the total area including turf and mature trees. Dominant tree types in the park are Ulmus (Elms), Corymbia (Gums) and Platanus (plane trees), with some Washingtonia (Petticoat or Desert Fan Palm), Angophora (Smooth-barked Apple Myrtle), Pyrus (Callery Pear) and Populus (Simon Poplar) all present. This park is regularly watered by City of Melbourne council.

Measurement fields was conducted to collect data by using hand-held type of Kestrol (4000 Series Weather & Environmental Metre) for air temperature. Mechanism was used gathering air 6 measuring times per one day seasonally, therefore this method means monitoring the variance in temperature during 24 hours. Eight points were selected as the total points in each period time starting from the edge of the park and extended to 915 m south of the park through Lygon Street. The achieving of precise reading was selected in one point for every spot with taking a long time varying between 3-7 minutes to obtain the constant and right reading. The first point was selected is the average for three points or depending on the time itself, distributing in the middle and edges of the park. The others were selected far away from the edge by variable distances, depending on the best sunny spots and, the procedure was used taking three or four readings in the spot square with 1 m diameter approximately. Beside this, all measurements were corrected with a ground meteorological station (Emerald Hill, Australia), which were taken immediately at the same time of filed measurements.

![Argyle Square](image)

Figure 1. Argyle square boundaries (source: Google maps)

3. Results

The first part of the results is calculating park cool island (PCI), which means the difference in air temperature between the park edge and the transect points toward the city center. The second part is calculating park cooling distance (PCD) from the park edge towards the city center.
3.1 PCI results
All PCI magnitudes were corrected by the nearest ground weather station (Emerald Hill Australia) during the same time of measuring by using application in a smart phone, which automatically update at every 5 minutes. The procedure of correction was taking temperature reading twice, at the start in the first point and in the last point and take the difference between them plus or minus. After that the correction has been made decreasing or increasing the temperature. Table 1 shows the results of period 2 (8–10) am in spring, which explain the process of correction data. Maximum PCI magnitude 2.9°C appeared during period 1 (4–6) am in spring, maximum PCI magnitude 7.1°C showed during period 2 (8–10) am in summer, maximum PCI magnitude 5.5°C showed during period 3 (12–2) pm in summer, maximum PCI magnitude 3.5°C showed during period 4 (4–6) pm in summer, maximum PCI magnitude 4.1°C showed during period 5 (8–10) pm in summer and maximum PCI magnitude 3.2°C showed during period 6 (12–2) am in spring. Average maximum PCI for all measuring periods in all seasons reached to 7.3°C.

3.2 PCD results
Maximum PCD magnitude reached to 915 m, during periods 1 (4–6) am, 2 (8–10) am, 3 (12–2) and 5 (8–10) pm in all seasons except winter. Maximum PCD magnitude reached to 915 m, during period 4 (4–6) pm in both summer and autumn. Maximum PCD magnitude reached to 915 m, during period 6 (12–2) am in both summer and spring. Average maximum PCD for all measuring time in all seasons varies between 310 m–915 m.

<table>
<thead>
<tr>
<th>Distance (m)</th>
<th>Air temp. (Emerald Hill station) °C</th>
<th>Corrected air temp. (2/31 = 0.064°C)</th>
<th>Corrected value</th>
<th>Corrected air temp. °C</th>
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</thead>
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<tr>
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<td>0</td>
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<tr>
<td>20</td>
<td>26.4</td>
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<td>915</td>
<td>27.4</td>
<td>31</td>
<td>31*0.064</td>
<td>25.416</td>
</tr>
</tbody>
</table>

Table 1. Air temperature correction of period 2 (8–10) am in spring

4. Discussion
4.1 PCI & PCD
This park showed a decreasing in air temperature (PCI) in the surrounding areas during all measuring times in the day and night as [25] used small parks to decrease temperature between 0.4°C – 0.8°C. The magnitude of PCI varied between the six measuring times across the entire year, the maximum cooling magnitude recorded during morning time between 8–10 am. After that PCI decreased at noon and after noon periods (12–6 pm) was decreased. Then PCI was increased during night time (8–10) pm but it decreased during mid night period (12–2 am), which continued to decrease the minimum magnitude of cooling at early morning period (4–6 am). In contrast, [26] pointed out PCI during night time higher than day time hours. The park features, traffic situation and vegetation cover make the difference between those studies and this study. In terms of seasons, summer and spring were the best seasons to combat urban heat island (UHIs). The other reason is [27–30] selected different park numbers and areas with
different methodologies. All of them recorded significant PCI magnitudes which varied between 1°C–3.5°C, however, they were clearly different in PCD magnitude.

Respective to measuring time periods, all measuring time periods displayed the same cooling distance. In terms of season, this park did not reach to the maximum distance (915 m) in winter at all measuring time, while all measuring times in summer extended to the maximum cooling distance.

![Figure 2](image-url)

**Figure 2.** Air temperature recordings during period 1 (4–6) am (A), period 2 (8–10) am (B), period 3 (12–2) pm (C), period 4 (4–6) pm (D), period 5 (8–10) pm (E), period 6 (12–2) am (F).
5. Conclusion
A small park can mitigate urban heat island (UHIs) in both cooling magnitude (PCI) and cooling distance (PCD). The effect of the park cooling to decrease temperature showed at all measuring time periods. In terms of seasons, summer showed higher cooling effect than the other seasons and winter was the lowest.

6. References


Temporal Patterns of Urban Heat Island Intensity in Adelaide, Australia

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Keywords: Urbanization, urban heat island, urban density, day time UHI, night time UHI

Urban Heat Islands (UHI), whereby urban areas are warmer than surrounding rural areas, are becoming an increasingly important phenomenon as global populations continue to urbanize. This study examines the degree and the features of the UHI in Adelaide, Australia. Over 17 years, hourly meteorological observations from four stations across Adelaide representing high, medium, low density urban sites and a rural site were examined to determine how UHI magnitude relates to urban density. The results show that the highest average UHI value (1.34°C) is found in the high density urban site while the lowest average UHI value (0.53°C) is found in the low density urban site. The high density urban site also exhibited a UHI for 72.5% of the data, in contrast to the low density urban site for which a UHI was only evident 53.2% of the time. Regression and trend analyses show that the continued urbanization of the medium and low density urban sites is causing their UHI intensities to increase over time while the highest density urban sites UHI magnitude has stayed steady.

1. Introduction

The majority of the words population lives in cities, and this proportion is increasing every year [1]. Although urban living has many advantages, it can also expose us to risks. One of these is a phenomenon called the Urban Heat Island (UHI) effect. It is characterised by higher temperatures in cities relative to their rural surrounds. This is an issue because increased temperatures are a leading cause of injury and even death around the world with heat being the cause of more fatalities than all other natural disasters combined. Increased heat in cities also results in increased energy use (for cooling) which speeds up global warming [2]. Higher urban temperatures in conjunction with increased temperatures from global warming have the potential to make cities in some parts of the world largely unliveable (with very high risks from heat related illnesses or death), especially for those who cannot afford air conditioning.

UHIs have been widely reported to exist in many areas around the world. However, much remains unknown about their magnitude, spatial and temporal distributions and whether (or how) they change over time. For example, although many studies on UHI intensity in different places have been conducted, they typically use a wide range techniques that confound attempts to generalise the results. In some cases, the intensity is assessed using air temperature, while in others, land surface temperature. These intensities can be vastly different. Further, in some studies UHI is reported as a maximum while in others, it is reported as an average, in both cases, the time frame of data collection can be as short as a few weeks or as long as several decades. Considering this wide array of studies, UHI intensity has variously been reported as 5°C in Rome [3], 6.5°C in Shanghai [4], 7°C in London,[5], and as much as 12°C in Lodz, Poland [6]. However, none of these studies has expressly considered how the magnitude of UHIs can vary over time and between places within a city as a result of differing meteorological conditions (causing differences in time).
and urban characteristics (causing differences between places) [7]. If, as has been suspected based on the work of a few previous studies, that UHI intensity may be heterogeneous in space and can change over time [8,9] then these single value UHI intensity results do not offer much use to urban planners or land managers seeking to mitigate this phenomenon.

In addition to these limitations, previous studies into UHI intensity have been typically limited temporal in temporal scope. Indeed, most previous studies have been limited to short periods of observation (a few days, a single month, a single season, or at most one year) and limited to a typical criteria condition that would be most suitable for UHI development (clear skies and less wind). In addition, the UHI effect has mainly been analysed using a limited number of locations within a city. Thus, few studies of UHI intensity have analysed datasets that have had both a high temporal resolution and have covered an extended period of time [10,11].

The purpose of this study is to redress these issues by exploring the UHI intensity in Adelaide, Australia over a long duration (17 years of hourly meteorological observations) from multiple sites representing different urban configurations (four stations representing a high-density urban site, a medium-density urban site, a low-density urban site and a rural site) and considering changes in UHI intensity over time. This study is novel in its approach, and represents one of the most spatially and temporally comprehensive studies of UHI intensity ever attempted. Knowledge of the spatial and temporal aspect of UHI intensity will allow city planners to be better manage for heat related impacts and design mitigation measures that can account for the most common urban heating levels.

2. Study site, data and analysis methods

Adelaide is the fifth most populous city in Australia with a population 1.3 million. It is located at the base of the Mount Lofty Ranges in South Australia. Adelaide’s climate is Mediterranean and it has around 2,500 hours of sunshine annually. Summers are hot with temperatures regularly exceeding 35°C and sometimes 40°C. In winter, temperatures range from 7°C to16°C. Rainfall is also most common in winter and the average rainfall is about 550 mm.

To investigate the variability of the Urban Heat Island phenomenon in Adelaide, hourly climate data measurements from four Bureau of Meteorology monitoring sites within and surrounding the core of the city were selected for inclusion in this study. These sites are sub-divided into a high-density urban zone, a medium-density urban zone, and a low-density urban zone with a rural site being used as a point of comparison to compute UHI values according to the representative area approach described by Oke et al. [12]. The locations and characteristics of the four sites included in this research and their categorizations are presented in Table 1. In addition to being located in important geographic locations representing different urban densities, these sites were also chosen for their long continuous records with each station having 15 or more years of hourly temperature data available. Another important consideration in site selection was to minimize elevation differences between sites as this can create temperatures differences between sites that are unrelated to urbanization. As can be seen in Table 1, all sites were less than 70 m of elevation to each other.

In most prior studies, two representative stations were used to calculate the UHI intensity (one urban and one rural) [11]. An alternative approach, used in a few studies, has been to use station averaged temperature for both urban and non-urban zone [13] or in a neighbouring zone [10]. This study adopts a
unique approach, in that it explicitly seeks to determine the relative UHI of different parts of a city by comparing UHI intensities between sites found within areas of different urban densities. The data used in the analyses extend from the beginning of August 1997 to the end of September 2015 for each of the selected stations. These data were subjected to two forms of analysis for the purpose of this study. First, UHI/UCI magnitude-frequency histograms were produced for each urban density site. These allow for the comparative strength and distribution of UHI intensity over the period of record to be determined. Next, the data were explored using time series analysis (regression and Mann-Kendall Trend tests) to determine whether the intensity of the UHIs for each urban density area have been changing over time.

<table>
<thead>
<tr>
<th>Study sites</th>
<th>Station ID</th>
<th>Coordinates</th>
<th>Elevation (m)</th>
<th>Urbanisation level</th>
</tr>
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<tr>
<td>Adelaide (Kent Town)</td>
<td>023090</td>
<td>-34.92, 138.62</td>
<td>48.0</td>
<td>High density urban</td>
</tr>
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Table 1. Site characteristics for the weather stations included in this study

3. Results and Discussion

Fig. 1 shows the frequency distributions of hourly UHI intensity for the selected urban (high, medium and low density) sites. The results show that all of the urban sites (high, medium and low-density) experience UHIs more frequently than UCIs although UCIs occur at least some of the time in all cases. The high-density urban site has the highest proportion of UHIs relative to UCIs (UHIs occur 72.5% of the time) site followed by the medium density urban site (UHIs occur 67.4% of the time). The lowest frequency of UHI occurrence is found at the low urban density site which experiences a UHI 61.4% of the time and a UCI 38.6% of the time.

A closer inspection of the UHI magnitudes of the high density urban site (Fig. 1.a) reveals that the vast majority of observed UHI intensities are between 0°C and 5°C with those between 1°C and 4°C most common. However, values between 5°C and 10°C also occur quite regularly with up to 5.2% of UHI intensities in this range. In terms of UCIs for this site, most of the values are less than -1.0°C and UCIs greater than -5.0°C are very rare. This demonstrates that not only are UHIs much more common than UCIs for the high density urban site but also that the intensity of UHIs when they occur are far stronger than the intensity of UCIs.

For the medium density urban site, the UHI intensity also falls generally between 0°C and 5°C with those between 1°C and 4°C most common (Fig. 1.b). However, the tail is shorter for this site than for the high density site with only 2.5% of the data higher than 5°C. UCIs are relatively common for this site, with 32.4% of all temperatures differences characterised by the rural area being hotter than the urban area. However, the magnitudes of the UCIs is typically low (rarely more than -2.0°C) with UCIs greater than -5.0°C being very rare.

For the low urban density site, UHI intensity falls generally between 0°C and 5°C with those between 1°C and 4°C most common (Fig. 1.c). However, the tail is longer than that for the medium density urban site with 4.5% of the data higher than 5°C. UCIs are common for this site, with nearly 40% of all temperatures differences characterised by the rural area being hotter than the urban area. Unlike the other two sites, In
addition, unlike the other two urban density sites, high UCI values also occur with some regularity, with 3.4\% of observations exhibiting UCIs of between -5°C and -10°C.

In addition to identifying UHI and UCI magnitudes and distributions relative to urban density, this study also seeks to determine whether UHI magnitudes are changing over time for each of the urban density sites. Results for the annual trends analysis are presented in Table 3. These results show that the UHI magnitudes for the high urban density site are not changing over time. This likely reflects the fact that this site has had a high degree of urbanisation for the entire period of record. While those for the medium and low-density urban sites are changing (increasing) over time.

**Figure 1.** Frequency distribution of UHI intensity for different density urban areas: (a) high density urban site; (b) medium density urban site; (c) low density urban site
These results can be explained by the growth of urbanisation (or changes in the built environment) in each site over the period of record. Data for this study spans a period from 1997 to 2015. Over this time, the high density urban site has been, and continues to be, a high density site. It has effectively reached peak-urbanisation and any changes that have occurred over this time generally reflect replacing like with like (i.e., any buildings removed are replaced with similar ones and vegetation cover remains roughly the same).

![Figure 2](image)

**Figure 2.** Statistical trend analysis (Mann-Kendall Trend) for different urban densities: (a) high density urban site; (b) medium density urban site; (c) low density urban site

In contrast, the medium and low density urban sites have experienced increased rates of urbanisation over this period. This has taken the form of more and higher density structures, and a reduction in vegetation cover corresponding with an increase in impervious surface cover. These results, then, illustrate how increasing urbanisation continues to increase UHI magnitudes until peak urbanisation is reached, at which point UHI magnitudes stabilise. This study represents the first instance where this phenomenon has been observed.
<table>
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<td>r² values</td>
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Table 3. Results of the statistical trend analysis tests for annual UHI intensity over the study period. The p-value and Sen’s slope are for a Mann-Kendall Trend test. The r² values are for a simple linear regression. For both the p-value and r², values indicated in bold are statistically significant.

4. Conclusion

A series of qualitative, statistical and visual analysis were performed to investigate the UHI characteristics (frequency and intensity) in Adelaide. This study included data from four meteorological stations representing different urban densities (high urban density site, medium urban density site, low urban density site and a rural site) over a period of 17 years (from August of 1997 to September of 2015). The results of this study show that the UHI phenomenon is quite common in Adelaide. Indeed, UHIs were found to occur between 61.4% (for the low urban density site) and greater than 72% (for the high urban density site) of the time. UCIs were both less common (temporally) and of lower magnitude than UHIs for all urban areas. In addition, the results of this study show that the continued urbanisation of the medium and low density urban sites is causing their UHI intensities to increase over time while the highest density urban sites UHI magnitude has stayed steady (most likely owing to the fact that it has reached peak urbanisation).

References


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Seasonal Patterns of Urban Heat Island Intensity in Adelaide, Australia

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2AL-Mustansiriyah University, Baghdad, Iraq
3Department of Climatology, Adam Mickiewicz University, Poznan, Poland
4School of Global, Urban and Social Studies, RMIT University, Melbourne, Australia

Keywords: Climate change, urban density, urban heat island, seasonal patterns

Many cities around the world are significantly warmer than their adjacent non-urban surroundings, a phenomenon known as the urban heat island (UHI) effect. In this study, hourly meteorological data over a 17-year period were examined for four weather stations (representing an urban density gradient) across the city of Adelaide, Australia to determine the spatial pattern of Urban Heat Island (UHI) intensity in different seasons. The results show that the average monthly urban heat island intensity seems to be higher during September and October, with the strength of the UHI dependent on the density of the urban environment (with high urban density sites exhibiting the highest UHI intensities). The data were also examined with respect to day-night variations. These data show that, in most cases, the seasonal UHI is strongest overnight and into the early morning, with the highest UHI intensity occurring at 22:00. In contrast, during the day, urban cool islands (UCIs) were often observed, especially in summer, spring and autumn. These results highlight the spatial and temporal variability of UHIs in cities and indicate that studies of UHI should take these factors into account to improve the efficiency of mitigation measures.

1. Introduction

Urban Heat Islands (UHIs) are a phenomenon ubiquitous in global cities. They occur when the temperature in the city is higher than those in surrounding, non-urban areas. This is important because an increasingly large proportion of the globes population are living in cities and because heat causes more fatalities than all other natural disasters combined. The likelihood of heat related fatalities will only increase in cities in coming years due to global warming, meaning that it is urgent that we understand this phenomenon to help mitigate its impacts [1,2].

Previous studies have suggested that the magnitude of UHI commonly ranges from 5°C to 10°C [3-5] but has also been reported to be as high as 14°C [6]. However, it is very difficult to ascertain the validity of these results as studies of UHIs often differ significantly in terms of the type of data used (air or surface temperature), the way UHI data are described (mean, maximum, minimum), data acquisition methods (weather station observations, mobile traverses or remote sensing data), the time span of the studies (several days to one or more years) and the choice of stations/locations within cities and in adjacent rural areas [6-10]. Moreover, previous researchers have suggested that UHI intensity may be heterogeneous in space and can change over time [11,12]. For instance, Memon and Leung [13] reported that mean UHI intensity in Hong Kong was 0.5°C in summer and 2°C in winter, but the maximum UHI could be as high as 10°C at any particular time. Meanwhile, Kim [9] suggested that night time UHI magnitudes were strong in autumn and winter and less pronounced in summer, while day time UHIs were similar across all seasons. This complexity with respect to times of day and seasonality requires additional research as any attempt
to mitigate UHIs must necessarily understand how they differ in intensity at different times and in different places.

This last issue, variation in UHI intensity within cities, has not been well studied by previous investigators. To redress this issue, this study will investigate spatial (along an urban density gradient) and temporal (including both day/night and seasonality) patterns in UHI intensity using *hourly surface* continuous temperature measurement *data over a 17-year period (from August 1997 to September 2015) for four weather stations across the city of Adelaide*, Australia. It is hoped that, by better understanding the temporal and spatial variations in UHI intensity across Adelaide, this phenomenon can be better managed to reduce the harmful impacts of elevated urban temperatures across the city.

### 2. Study site

Adelaide is located on the south west coast of Australia at 34.5°S latitude at the base of the Mount Lofty Ranges. Adelaide is the capital of the state of South Australia (SA), and covers a land area of approximately 3,260 km² and has a population of approximately 1.3 million people (2016). Adelaide’s climate is Mediterranean with around 2,500 hours of sunshine annually. In summer, temperatures can exceed 40°C with temperatures above 30°C quite common. In winter, the mean temperature range 7°C to 16°C. Rainfall comes mainly in the winter, with an annual average about 550 mm. This study includes data collected from four weather stations. The properties of these stations are summarised in Table 1.

<table>
<thead>
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<td>Medium density urban</td>
</tr>
<tr>
<td>Adelaide Airport</td>
<td>023034</td>
<td>-34.95, 138.52</td>
<td>2.0</td>
<td>Low density urban</td>
</tr>
<tr>
<td>Roseworthy</td>
<td>023122</td>
<td>-34.51, 138.68</td>
<td>65.0</td>
<td>Rural</td>
</tr>
</tbody>
</table>

**Table 1.** Site characteristics for the weather stations included in this study

### 3. Data and analysis methods

Hourly air temperature data were obtained from four Bureau of Meteorology weather stations in the Adelaide Metropolitan area from 1997 to 2015. The selected sites represented different density urban sites. The magnitude of the UHI was computed as the difference in air temperature between each urban site against a rural site. The air temperature differences above or below 0°C represent the occurrence of an UHI (urban site warmer than rural site) or an UCI, respectively. This study considers how UHI intensity varies spatially (between different urban densities) and temporally (seasonally and day/night). To accomplish this, graphical representations of the data are used in association with summary statistics.

### 4. Results and Discussion

The results of this study show that the UHI phenomenon is evident in Adelaide with the highest annual average UHI intensity occurring in the high-density urban site (Adelaide—Kent Town) with an average UHI magnitude of 1.34°C relative to the rural site (Roseworthy). In contrast, the UHI intensity of the medium density urban site (Parafield) is 0.84°C and for the low density urban site (Adelaide Airport) the average UHI intensity is 0.53°C. These results illustrate the importance of urban density on the magnitude of UHIs.
in Adelaide. However, to more fully understand the nature of Adelaide’s UHI, it is necessary to explore how it varies temporally as well. This facet of Adelaide’s UHI will be explored in the subsequent sections.

4.1 Monthly Variations in UHI Intensity
This part of the analysis considers how UHI intensity varies on a monthly basis for each urban density site. The results from this analysis are presented in Fig. 2. For all three urban density sites, the average monthly urban heat island intensity is highest in August, September and October and relatively low between November and May. The highest monthly UHI intensities are 2.26°C, 1.70°C and 1.84°C for the high, medium and low urban density sites, respectively, all of which occur in September. For all months, UHI values are positive for the high and medium urban density sites. However, the low density urban site exhibited a negative average (also known as a UCI) during January and February. The results, also show that the high urban density site has a UHI intensity that equals or exceeds those of the medium and low density urban sites in all months. However, the medium and low density urban sites each have periods where their UHI intensity is higher than the other. The medium density urban site has a higher UHI intensity than the low urban density site from October to April while the opposite is true from May to September.

Next, the data were further subdivided to consider day-night UHI intensities within each month. These data are presented in Fig. 2. For all urban densities across all months, the night time UHI is higher than the day time UHI, although the strength of the UHI is dependent on the density of the urban environment (with high urban density sites having UHI intensities equal or greater than the other two urban densities in all cases). Positive UHIs are evident for all urban density sites during the night for all months of the year and during the day for all months except those between November and March where UHI values were near 0 or, in some cases, UCIs were present (especially for the low urban density site where quite large UCI values occurred during the day between November and March). The higher UHI values observed overnight are consistent with previous studies that showed similar patterns [e.g. 9] and this can be attributed to the higher thermal admittance of artificial materials in built up areas relative to those in natural areas.

4.2 Seasonality of Day and Night UHI Intensity
To further understand the temporal pattern of UHI intensity, the data were next considered by season (Tables 2 and 3). This includes a consideration both of amalgamated daily data (Table 2) and day and night
data (Tables 3) by season. For the amalgamated daily data (which includes all observations in each season), there were clear seasonal variations in UHI intensity between seasons and between urban densities. The data suggest that for different density urban sites (high, medium, and low) the average seasonal UHI values are strongest in spring (1.57°C, 1.30°C, 1.01°C for the high, medium and low urban densities respectively) which is characterised by weak winds and low cloudiness, and lowest in summer (0.82°C, 0.77°C, -0.27°C for the high, medium and low urban densities respectively).

The mean, maximum and minimum day and night UHIs in each season are presented in Table 3. In terms of mean UHI intensity, night time UHIs are higher than daytime UHIs for all site comparisons in all seasons, however with differing strengths for the various density urban environments (highest for the high density urban site). Next, the results show that the highest absolute maximum seasonal UHI occurs during the daytime in autumn with magnitudes as high as 14.4°C in the low density urban site. In contrast, the absolute lowest seasonal UHI (really a UCI) occur during the daytime in summer with a magnitude of -14.4°C for the low density urban site. It is interesting to note that both the highest maximum and highest minimum UHIs (and UCIs) occurred in the low urban density site even though this site generally displays the lowest average UHI intensities across the entire year. Another interesting observation is that summer days for all urban density sites exhibit an average UCI while spring nights exhibit the highest average UHIs.

![Graph showing Monthly day and night UHI intensity for different urban densities. High density urban site (H), medium density urban site (M), low density urban site (L).](image)

The strong relationship between UHI magnitude and urban density can, at least partially, be attributed to the nature of the built environment (including both the urban form and the materials that are found there). This includes the lack of vegetation and water bodies in urban environments (which would serve to cool them via evapotranspiration), the urban canyon effect and a high thermal admittance in urban areas which tend to trap heat causing these areas to warm up more than rural areas [14]. Another factor that may influence the UHI magnitude in urban areas is anthropogenic heat, and this parameter is one that is likely to vary not only with urban density but also with time of day and seasonality as it strongly relates to the use of energy for heating and cooling.
<table>
<thead>
<tr>
<th>Study sites</th>
<th>Mean seasonal UHI (°C)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Summer</td>
<td>Autumn</td>
<td>Winter</td>
<td>Spring</td>
</tr>
<tr>
<td>High density urban</td>
<td>0.82</td>
<td>0.84</td>
<td>1.41</td>
<td>1.57</td>
</tr>
<tr>
<td>Medium density urban</td>
<td>0.77</td>
<td>0.78</td>
<td>0.82</td>
<td>1.30</td>
</tr>
<tr>
<td>Low density urban</td>
<td>-0.27</td>
<td>0.39</td>
<td>1.01</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Table 2. The mean UHI (°C) of each season in the selected high, medium and low density urban sites against rural site over the study period.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Season</th>
<th>Day/Night</th>
<th>High urban density</th>
<th>Medium urban density</th>
<th>Low urban density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean UHI (°C)</td>
<td>Summer</td>
<td>Night</td>
<td>1.76</td>
<td>1.51</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>-0.35</td>
<td>-0.06</td>
<td>-1.88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Night</td>
<td>1.46</td>
<td>1.07</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>0.15</td>
<td>0.38</td>
<td>-0.44</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>Night</td>
<td>1.58</td>
<td>0.88</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>1.17</td>
<td>0.71</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>Winter</td>
<td>Night</td>
<td>2.66</td>
<td>2.06</td>
<td>2.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>0.48</td>
<td>0.54</td>
<td>-0.41</td>
</tr>
<tr>
<td>Maximum UHI (°C)</td>
<td>Summer</td>
<td>Night</td>
<td>12.0</td>
<td>11.8</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>10.0</td>
<td>10.1</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Night</td>
<td>10.2</td>
<td>9.3</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>10.2</td>
<td>8.9</td>
<td>14.4</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>Night</td>
<td>9.8</td>
<td>9.4</td>
<td>9.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>9.2</td>
<td>8.5</td>
<td>9.2</td>
</tr>
<tr>
<td></td>
<td>Winter</td>
<td>Night</td>
<td>13.4</td>
<td>10.9</td>
<td>11.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>11.0</td>
<td>10.5</td>
<td>10.2</td>
</tr>
<tr>
<td>Minimum UHI (°C)</td>
<td>Summer</td>
<td>Night</td>
<td>-11.0</td>
<td>-11.8</td>
<td>-10.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>-11.8</td>
<td>-9.9</td>
<td>-14.4</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>Night</td>
<td>-0.3</td>
<td>-7.3</td>
<td>-6.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>-8.7</td>
<td>-7.2</td>
<td>-11.6</td>
</tr>
<tr>
<td></td>
<td>Winter</td>
<td>Night</td>
<td>-5.1</td>
<td>-6.3</td>
<td>-6.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>-4.1</td>
<td>-5.8</td>
<td>-6.6</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>Night</td>
<td>-6.2</td>
<td>-6.5</td>
<td>-8.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Day</td>
<td>-9.2</td>
<td>-7.5</td>
<td>-13.1</td>
</tr>
</tbody>
</table>

Table 3. Summary statistics for day/night UHI intensities between seasons for high, medium and low urban density sites.

5. Conclusion
In this paper, the spatial and temporal variations in UHI intensity were determined for the City of Adelaide, Australia. To achieve this, hourly temperature data from four metrological weather stations, representing different urban densities were examined. The results of this study show that urban density has a strong influence on UHI magnitude with the highest UHI intensities occurring in the high density urban site and
the lowest occurring in the low urban density site (which also tended to exhibit UCIs for much of the year). The study also revealed that time of year (expressed as month and season) and time of day (expressed as day or night) had a strong influence on UHI intensity. The highest UHI values tended to occur in September and October (spring) while the lowest tended to occur in summer. Night time UHIs were also found to be higher than those in day time with all urban densities exhibiting UCIs, on average, in summer. This improved understanding of the spatial and temporal patterns of UHI intensity can be used to better manage the impacts of UHIs and to design mitigation strategies appropriate to the level of risk.

References


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Composite Repairs to Bridge Steels

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**Keywords**: aging bridges; ASTM E647-13a; Composite repairs; corrosion; fatigue crack.

Steel structures are widely used in road and railway infrastructure, housing, mining, transportation, etc. The combination of environmental conditions (temperature and moisture) and operational loads can result in significant degradations in static strength and durability. Hence the focus of this research is on means for the rehabilitation of steel structures. Standard methods for rehabilitating steel structures, which involve drilling holes to attach repairs in regions that are known to be critical, have several disadvantages. Consequently attention has recently focused on the use of bonded composites doublers as possible repair alternatives. Whilst the use of bonded composite doublers has been shown to be particularly effective in repairing aluminium and steel airframes the limitations of this technique, as applied to civil infrastructure, are still being determined. This research investigates the use of externally bonded CFRP patches to rehabilitate steel structures. It is shown that the role of environmental exposure needs further research so as to determine its effect on composite repairs in realistic operational conditions. As such we examine bond durability under the simultaneous extreme weather conditions and fatigue

1. Introduction

Many Bridges made of steel have been degraded and lost their safety over the last few decades. Transportation for America subsequently conducted an analysis of the US National Bridge Inventory and reported that one in nine U.S. bridges were rated as being structurally obsoleted. Indeed, the collapse of number of aged bridges around the world resulted in adopting various maintaining and strengthening approaches. In July 2008, the US House of Representatives introduced the Bridge Life Extension Act of 2008 by US Rep. Michael Conway (R-TX11), after the collapse of the I35W bridge in Minneapolis, USA. It has long been known that the corrosion has a marked effect on the steel bridges which severely impact their integrity.\cite{1}. Bridges made of steel consists more than 43% of the US bridges as reported recently, accordingly, rehabilitation of these structures has been attained many thorough and comprehensive research studies, which have been concluded that advanced composite materials present superlative potential for their strengthening\cite{2}. The applications of fibre-reinforced polymers (FRP) to reinforced civil infrastructures have been widely attracted since the late 1980s\cite{3}. It has been proven theoretically that bonding CFRP to steel structures enhance their strengthening effectively\cite{4, 5, 6}. However, few studies exist on bond durability and the associated failure mechanisms arising from the simultaneous effect of extreme weather conditions and fatigue. As such the main aspect of the present work focuses on developing an experimental facility to investigate the simultaneous influence of loading, moisture and temperature on the CFRP/steel system. In this article to date we have developed a new purpose driven environmental test capability.
2. Durability Study

2.1. Set-up an Experimental Design:
The test design developed to investigate the combined effect of the environment condition and the mechanical loading on the bonded CFRP steel joints is shown in Figure 1. This design provides exposure to the corrosive environment while the specimen undergoes mechanical loading, as it established with regard to [7], see Figure 1(a). Two corrosion chambers were used. The main chamber, which was made of acrylic, performed two crucial jobs; One is holding the solution prepared in particular to this test which was 3.5% NaCl artificial sea water at a maximum temperature of 60 °C. This chamber also contains a metal grip which penetrates through its bottom and connects the specimen to the lower grip of the fatigue machine as this is its main other job. This chamber was not covered at the top as this allows the specimen to be directly connected to the upper grip of the machine. It also has two apertures used for solution inlet and outlet respectively. Circulation of the solution was continuously needed to maintain the temperature as same as the solution in the water path as indicated in Figure 1(d), though the latter was provided with a heater and a pump to circulate the water and control its temperature inside the chamber. An additional chamber, which was made of metal, embraced the first chamber.

![Figure 1: Experimental design for the simultaneous effect of environment and load: a) A schematic view of the design set-up, b) experimental set-up of the durability test c) a view of the secondary chamber of the rig, and d) the water bath which contains water pump and heater.](image)

The role of this second chamber, which is labelled number 7 in Figure 1(b), was to grantee no solution leakage and to support the whole rig inside the upper and lower machine grips. In order to enable the fatigue testing to be performed normally while the specimen is immersed totally in the environment, an additional grip was used to handle the specimen from one end and to connect it to the fatigue machine from the other. The grip that was submerged completely in the environment was made of stainless steel so as to resist the anticipated corrosion, see Figure 1 (c). The test specimens were designed to fit inside
the rig and as such were designed to be maximum 400 cm long. The specimens also had a hole at one end in order to be gripped inside the chamber which is labelled 4 in Figure 1. Thus the bearing capacity of this member was a critical design consideration.

2.2. The Moisture Uptake Investigation For Normal Modulus CFRP Plates (Diffusivity Rate):
Prior to onset the bond durability experiment program, a crucial consideration should be taken into account. For the double lab joints specimen configuration which has the top two outer surfaces of CFRP, so as when this specimen is exposed to the environment (the moisture) in the time of testing, this performing can endure inconsistency, as the moisture might not approach to the bond interface (the interface in between the composite and the adhesive). However, it was found that for double lab joints specimens, as steel plate reinforced with normal modulus CFRP, bond failure happened at the interfaces in-between the composites/steel [8]. Consequently, the time required for moisture to reach the interfaces in-between the composites/adhesive is an important consideration? To this end, a diffusivity rate experiment of CFRP materials was performed in order to (approximately) estimate the time required for the moisture to be absorbed by the composite and reach the composites/adhesive interface. This was determined as per the standard test as detailed in ASTM D5229. To this end 10 CFRP plates specimens of the dimensions (50 x 200 x 1.2 mm) were prepared. These specimens were first dried in an oven for 24 hours and weighed, before being immersed in the environment which was 3.5% NaCl artificial sea water. Two different exposure temperatures 20 and 50 °C, were utilised, the measurements were done at various time intervals every 24 hour for the first week and every 2 days for the next 12 days. At each measurement interval the specimens were removed from the water tank and extra water on the surface was dried using clean tissues. The specimens were then they were weighed using a digital balance to measure the weight gain every exposure period. The change in weight as a percentage was plotted against the square root of time (√t), where t is the time in days. Once the moisture uptake equilibrium occurs then this is the time that the sea water requires to approach the bond interface see Figure 2. Accordingly, and by using equation [1] the diffusivity rate of normal modulus CFRP material has been calculated.

\[
D = \pi \left( \frac{h}{4M} \right)^2 + \left( \frac{M^2 - M_1}{\sqrt{12} - \sqrt{14}} \right)^2 + \left( 1 + \frac{h}{l} + \frac{h}{w} \right)^{-2}
\]  

(1)

![Figure 2: Moisture uptake curve for the CFRP laminates immersed in sea-water](image-url)
Table 1: Characteristics of moisture uptake and the diffusivity coefficient of normal modulus CFRP laminate.

<table>
<thead>
<tr>
<th>The Environment</th>
<th>Temperature (°C)</th>
<th>Maximum moisture Uptake (%)</th>
<th>Coefficient of Diffusion (mm²/s) x 10⁻⁹</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 %wt NaCl</td>
<td>20</td>
<td>1.03</td>
<td>2.52</td>
</tr>
<tr>
<td>3.5 %wt NaCl</td>
<td>50</td>
<td>1.7</td>
<td>3.00</td>
</tr>
</tbody>
</table>

2.3. The Bond Durability Experiment

2.3.1. Test Configuration and Material Properties

CFRP/steel double lap joints or double overlap fatigue specimen (dofs) with dimensions shown in Figure 3 were fabricated. These specimens consisted of two pieces of 350 grade mild steel bonded to normal modulus (E = 200 GPa) CFRP outer adherends. The dimensions of the steel inner adherends were: 180 mm long, 50 mm width and 8 mm thick. The size and configuration of specimens were designed according to AS1391 specifications (2001), and designed to fit in the designed rig see Figure 1, and be tested under fatigue load. These two outer adherends were a layer of CFRP composite, MBRACE laminate normal modulus (E = 210,000 MPa), 200mm long, 50 mm wide and 1.4 thick. A CFRP laminate was used since it is perhaps the most commonly used composite material used in the repair of steel infrastructure. The epoxy resin adhesive (Araldite 420 A/B) was used as the adhesive system in this test. Araldite 420 has widespread appreciation in bonding various structural substances such as metal, wood, rubber, composites and many plastics. This is attributed to its typical properties, namely, design flexibility, extremely tough but resilient, relative rigidity, and its cost-effective characteristic, in addition to its room temperature curing property [10]. The manufacturer outlines that the rigid CFRP plate has a tensile strength and elastic modulus of 3,300 MPa and 210 GPa respectively. The Poisson’s ratio was assumed to be 0.35, which is a typical value for this type of adhesive [11].

![Figure 3: Schematic design of the CFRP/steel double lap joint.](image-url)
2.3.2. Specimens Preparation

The dofs specimen geometry was originally developed [12], since it closely simulates the stress state in both the repair and the adhesive associated with a composite repair to a cracked metal structure.

As recommended in [13-15] surface sandblasting was used as a mechanical surface preparation technique to roughen steel surface. Surface roughness helps to increase the surface area, which enhances adsorption and interlocking, thereby facilitating adhesion and bonding. The steel surface was Abrasive sandblasted by subjecting it to an accelerating media (16-grit sand) through a blasting nozzle by means of compressed air. (Note that the gun should be held away about 5 cm from the surface at 450 angle.) The (roughned) steel surface was then cleaned first by compressed air to remove the dust sand particles and secondly with acetone using clean brush. Roughness the surface (sandblasting) and then wiping it with chemical solvent (acetone) can produce contamination free surface, chemically active and fresh surface, and an interface resistant to hydration due to solvent treatment.

Afterward, a layer of Araldite adhesive which comes of two parts A & B and mixed before using with the percentage 10-4 was applied to patch a same surface size of CFRP laminate (200 x 50 mm) on the top directly. Finally the surface was rolled using plastic roller to remove the air bubbles and the extra adhesive. This was done to obtain an even and thin layer of epoxy. The specimen was subsequently left for curing at room temperature for at least 15 days.

2.3.3. Specimens Exposed Process:

1. Control Specimens:

After curing, 18 specimens were used to determine the strength of the bond in dofs, these specimens act as control specimens which were tensile static tested in three different temperatures (20, 40, 50 °C). 9 specimens were pre-immersed in sea water at the same testing temperatures (20, 40, 50 °C) for almost two weeks. This was done with regard to the moisture uptake equilibrium of normal modulus CFRP laminate as were calculated in 2.3, another 9 specimens were tested directly without any pre-immersion.

All specimens were loaded at a rate of 2mm/min, to exclude statistical error 3 control specimens were tested at each exposure temperature in a 3.5 % by weight NaCl artificial seawater.

Two dummy specimen with 4 attached thermocouples were used to measure the temperature in the interface between the CFRP and the adhesive see figure 3 as they immersed in the chamber where the real specimen placed (dof specimen which will be tested). After about 25-30 mins of soaking the dummy specimen in the environment, the chamber components reach the required temperature. Then dof specimen was placed and submerged into the chamber.

2. Fatigued Specimens:

Subsequently 12 other specimens were fatigue loaded under constant cycles, up to a maximum load of 20 % of the control specimen’s ultimate strength, before being static tested until failure. 6 specimens were sustained pre-exposure scenarios at three different temperatures (20, 40, 50 °C), while 6 others were directly tested under fatigue load which were companied with the exposure of the environment under same three temperatures. For each temperature two specimens were tested for duplicate purpose. Fatigue test was accomplished using MTS Instron machine with load cell capacity of 100 kN . For all tests, specimens were yielded a uniform fatigue load with 10 Hz frequency, 0.1 stress ratio and 2 million cycle. After specimen being fatigued then it was subjected to static load until failure in order to determine the
effect of the simultaneous exposure of corrosion and fatigue on the ultimate strength of the bond which as the results shown in table 2.

2.3.4. Results and Discussion:

1. Control Specimens:
For the control specimens, triple tests for each environment temperature were tested under the simultaneous exposure of environment and static load. Their ultimate tensile strengths are shown in Table 2. Figure 4 shows that the failure loads decreased as temperatures increased. The failure mode that the specimens exhibited at all temperatures were both cohesive and adhesive failure as shown in Figure 5. Unfortunately, this means that the surface preparation was inappropriate.

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Ultimate strength (kN) (Fave)</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>89</td>
<td>0.011</td>
</tr>
<tr>
<td>40</td>
<td>56</td>
<td>0.07</td>
</tr>
<tr>
<td>50</td>
<td>44</td>
<td>0.12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Ultimate strength (kN) after immersion for 2 weeks (Fave)</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>78</td>
<td>0.022</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
<td>0.06</td>
</tr>
<tr>
<td>50</td>
<td>40</td>
<td>0.14</td>
</tr>
</tbody>
</table>

Table 2: Ultimate loads of different temperatures in two various exposure to environment CFRP laminate repaired steel plates.

Figure 4: Failed surfaces for a) environmentally conditioned specimens and b) specimens without pre-exposure
2. Fatigued Specimens:

After fatigue loading, specimen’s residual tensile strengths. Comparing these to the original 20, 40, 50 °C control strengths gave ratios that best described the strength loss due to the applied conditioning. These ratios allow easy comparisons to be drawn between specimens and variables, the lower the ratio the more degradation caused during testing. The fatigued specimens witnessed premature failure in all temperature which were subjected to 0.2 of its ultimate strength. These specimens then tested until failed. The reduction of the bond strength due to fatigue loading was 0.95, 0.92, 0.94 and 0.96, 0.97, and 0.95 for 20, 40, 50 °C, for both procedures without and with pre-exposure respectively see figure 5 which shows the fatigued specimens in these three temperatures. Complete raw data for all specimens can be found in Table A1 in appendix A. In these fatigue load specimens the failure mode were both cohesive and adhesive failure which is the same as the specimens tested under the combined effect of environment and static load, although the environment temperature reduced the strength of the bond significantly.

The tensile ultimate strength of the bond of three exposure temperature were investigated for two procedures with and without pre-exposure to environment for two weeks see figure 6. This study led to realisation that alternative surface preparations were needed.

A comparison between the tensile ultimate strength of the bond tested under different environment exposure procedures was conducted to investigate the effect of the combined exposure of environment and load. Four different procedure were examined see figure 7, [16] exposed the specimens to temperature effect simultaneously with tensile loading, [17] used the pre-exposure to environment and temperature which is prior to the test, these two results were compared with the recent results which adopted the simultaneous exposure of environment, temperature, and load for two procedures of subjecting to environment. It is clear that the simultaneous exposure is the most severe procedure which effect the ultimate strength of the bond dramatically, although the pre-exposure to environment scenario was the lowest ultimate strength.
3. Conclusions

From these various investigations several important conclusion can be made:

1. The CFRP bonded steel specimens tested in an aggressive environment showed a significant reduction in bond strength. However, as a result of this test program and a subsequent literature survey future surface preparations can be adopted the industry standard, which involves grit blasting and the use of the USAF/Boeing sol gel process.

2. The failure modes involved a combination of cohesive and adhesive failure which assist the first recommendation of the importance of using the interior surface preparation which is recommended in the literature.

   As such it is clear that an effective bonding process that can withstand combined fatigue-environment is required. To this end tests will also be conducted using the Boeing-USAF surface preparation procedures viz: the use of grit blast and solgel.

3. For specimens that had been environmentally preconditioned the combined effect of high temperature, sea water and load resulted in the highest reduction in the ultimate tensile strength of the joints which is due to the glass transition temperature of the adhesive, therefore a new adhesive should be introduced in the future work.

4. Inspect the bond quality reveals that the bond was kissing bond (touching but no bonding) therefore refine bonding process is required to get good bonding.

5. Part of BAPST formula is invalid.

4. References


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Openings effect on the performance of reinforced concrete deep beams

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Keywords: deep beam, large web opening, diagonal cracks, ultimate load

Four simply supported reinforced concrete deep beams with and without large web openings have been cast and tested up to failure under one-point load. These beams have been tested to investigate the effect of increasing opening length (i.e., opening ratio) with fixed openings location and depth on the performance of reinforced concrete deep beams. In which the opening ratio is defined as the opening length divided by the overall depth of the section. Three of these beams having opening ratio of (0.4, 0.6 and 0.8). While, the remaining beam was solid and taken as a control beam. The results showed that increasing opening ratio caused decreasing in first cracking and ultimate loads and increasing the mid-span deflection. Also, increasing opening ratio induced early appearance of first diagonal cracks as well as increasing cracks width. The maximum percentages decreasing in first cracking and ultimate loads were (65.0 and 50.0) % respectively for beam with opening ratio equal to 0.8 compare with the control beam. While, the maximum percentage increasing in mid-span deflection was about 190.0 % for same opening ratio.

1. Introduction

Beam in which its clear span is less than four times the overall depth of the section is classified as deep beam [1]. Deep beams are often being used in reinforced concrete (RC) structures such as diaphragms and transfer girders. Predicting the behaviour of deep beams with web openings is important due to the strong development of construction work which is used for doors, windows and accommodate fundamental services.

Mansur and Tan [4] classified the web openings in RC beams as small and large openings. They suggested that the opening could be considered small if the ratio of web opening depth (h) to the overall depth of the section (H) is less than 25%. Otherwise, the opening could be considered large [4]. It was noticed that the implication of web openings and increasing of their sizes would effect the ultimate shear capacity of deep beams due to reducing of concrete mass acting in the compression zone and the opening works as a stress raiser for shear crack diffusion [2].

Previous studies which carried out by various researchers such as (Yoo and et al. [6] In 2007, Campione & Minafò [3] In 2012, Yoo and et al. [5] In 2013) had examined the ultimate load-carrying capacity of RC deep beams with web openings. Yoo and et al. [6] studied the behaviour of pre – stress RC deep beam with web opening. They found that the mode of failure of deep beams with web openings was like as solid concrete deep beams with minor cracks occurring below and above the outer edge of openings.

Campione & Minafò [6] studied the influence of small circular web opening on RC deep beams. The variables were the amount of reinforcement and the location of web opening. They found that the effect of web opening was depend on its position and the importance of the presence of reinforcement was depend on its arrangement. Yoo and et al. [5] studied the behaviour of high strength RC deep beams with
varies location of small web opening. They found that the increasing in concrete compressive strength \( (f'_c) \) has a significant effect on improving the behaviour and load carrying capacity of RC deep beams with web opening.

It can be notice that most of these researches deal with small web openings. Also, according to the review of published research on RC deep beams with web openings, most of these researches were focused on the effect of compressive strength of concrete, size and locations of web openings and the shear span to depth ratio. However, the influence of increasing web opening sizes (i.e., opening length) with fixed location and fixed depth was not taken into consideration. Therefore, there is a growing need for an accurate study into RC deep beams with large web openings taken into account the effect of increasing the web opening sizes as its presented in this research.

The main objective of this research is to investigate the behaviour of RC deep beams with different sizes of large web openings (i.e., different opening ratios) and predicting the change in steel stresses due to presence of large web openings.

2. Experimental work

2.1. Deep beams description

Four RC deep beams with and without symmetric of large web openings have been cast and tested up to failure. All beams having the same dimensions of \((150 \times 400 \times 1600)\) mm. The main variable in this research is the web opening size (i.e., opening ratio).

Three opening ratios have been considered in this research which are \((0.4, 0.6 \text{ and } 0.8)\). While, the remaining beam was without openings as a control beam. The opening height \((h)\) was kept constant of \(160\) mm for all tested beams (i.e., \(0.4 \text{ H}\)). Fig. (1) shows the layout of testing deep beams. While, full description for testing beams is listed in Table (1).

<table>
<thead>
<tr>
<th>Deep Beam Designation</th>
<th>Opening Ratio</th>
<th>Opening Dimension ((h \times w)) (\text{mm})</th>
</tr>
</thead>
<tbody>
<tr>
<td>R – DB – Zero</td>
<td>0</td>
<td>--</td>
</tr>
<tr>
<td>R – DB – 160</td>
<td>0.4</td>
<td>((160 \times 160))</td>
</tr>
<tr>
<td>R – DB – 240</td>
<td>0.6</td>
<td>((160 \times 240))</td>
</tr>
<tr>
<td>R – DB – 320</td>
<td>0.8</td>
<td>((160 \times 320))</td>
</tr>
</tbody>
</table>

Table 1: Nomenclature of deep beams

![Figure 1: Layout of a typical tested beam (all dimension are in mm).](image)
2.2. Specimens details
All tested deep beams having the same design for interior reinforcement. The longitudinal reinforcement is designed with (3Ø16) mm and (2Ø12) mm for bottom and top reinforcement respectively, (Ø6 @ 80) mm is used for stirrups. While, (4 Ø6) mm (2 at each side face of beams cross section) is used for skin reinforcement as shown in Fig. (2). All previous proposed reinforcement aspects have been checked with the (ACI 318 -14) Code [1] except skin (nominal) reinforcement due to presence of web openings. All supporting plates used to support the beams as well as the bearing plate which is used under the applied load having a dimension of (150 x 60 x 10) mm.

![Figure 2: Typical reinforcement layout for tested deep beams (all dimensions are in mm).](image)

2.3. Materials properties
The components of normal weight concrete that used to cast the specimens in the present study are: cement, fine aggregates, coarse aggregates and tap water. For all tested deep beams, the concrete compressive strength $f_{c}^c$ at 28 days was 30 MPa. Medium workability with a slump of (75 – 100) mm was achieved after several trial mixes. Deformed steel bars have been used for reinforcement. The yield stress for steel bars having a diameter of (16, 12 and 6) mm are (660, 520 and 520) MPa respectively.

2.4. Fabrication and Casting of specimens
To fabricate each specimen, steel bars were cut to the required length, then strain gauges were fixed at the centre of the longitudinal bottom reinforcement as well as at the left and right stirrups closed to opening edges as shown in Fig. (3-a). To fabricate web opening, Styropor blocks were prepared according to required opening dimensions for each beam and inserted inside the steel cage at the required opening location.

Before casting, the moulds were lubricated with oil for easy removal of specimen. The concrete was levelled when the adequate amount of concrete was placed into the mould so that the specimens were kept at the same height. Fig. (3-b) shows the deep beam specimens after completing casting and curing process.
2.5. Testing procedure

After completing 28 days of curing, the specimens were painted with white colour to identify cracks during the loading process. Specimens were then placed inside the testing frame and adjusted so that the centre line of point loading, supports and dial gauge were fixed in their correct locations as shown in Fig. (4).

3. Results and discussion

3.1. First cracking loads

It was observed from the experimental test that the diagonal cracks were formed first by 45° at the upper corner of web openings or from supporting plate. While, the first flexure cracks were formed with increasing load increment. Table (3) summarizes the first diagonal cracking loads for all tested deep beams. From this table, it can notice that the presence of web opening lead to early appearance of diagonal cracks. The percentages decreasing in first diagonal crack loads as compared with the control beam (i.e., R – DB – Zero) were (30.0,50.0 and 65.0) % for deep beams with opening ratios (0.4,0.6 and 0.8) respectively.
Table 3: Effect of presence of web opening on first diagonal crack loads.

<table>
<thead>
<tr>
<th>Deep Beam Designation</th>
<th>Opening Ratio</th>
<th>First Diagonal Cracking Loads kN</th>
<th>% Decreasing In First Diagonal Cracking Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>R – DB – Zero</td>
<td>0</td>
<td>100</td>
<td>---</td>
</tr>
<tr>
<td>R – DB – 160</td>
<td>0.4</td>
<td>70</td>
<td>30.0</td>
</tr>
<tr>
<td>R – DB – 240</td>
<td>0.6</td>
<td>50</td>
<td>50.0</td>
</tr>
<tr>
<td>R – DB – 320</td>
<td>0.8</td>
<td>35</td>
<td>65.0</td>
</tr>
</tbody>
</table>

3.2. Crack pattern and failure mode

Diagonal cracks or shear cracks caused failure in most cases for testing deep beams. It can be noticed that diagonal splitting failure and shear – compression failure are the modes of failure for testing deep beams. In which the first mode referred to failure due to diagonal crack at corner of web openings which developed towards support and loading point. While, the second mode referred to diagonal crack developed in shear span and crushing in compression due to high stress in compression zone. Fig. (5) shows the crack pattern at ultimate load for testing deep beams.

![Crack pattern at ultimate load for tested deep beams.](image)

3.3. Load – deflection response

Central deflection has been recorded for each deep beam during the test by using dial gage located at the mid span of deep beams. Fig. (6) shows the effect of the presence of web openings on load – deflection curves for testing deep beams. It can be noticed from this figure that the presence of web openings caused decreasing in ultimate load and increasing in mid span deflection. Table (5) summarizes the ultimate load and mid span deflection for all tested deep beams. It could be notice from this table that the percentages decreasing in ultimate loads as compared with the control beam were (26.7, 43.3 and 50.0) % for deep beams with opening ratios (0.4,0.6 and 0.8) respectively. While, the percentages increasing in mid span deflection were (37.9, 79.3 and 189.6) % for same opening ratios.
Table 5: Effect of the presence of web opening on ultimate loads and mid span deflection response.

<table>
<thead>
<tr>
<th>Deep Beam Designation</th>
<th>Opening Ratio</th>
<th>Ultimate Load (kN)</th>
<th>% Decreasing In Ultimate Load *</th>
<th>Mid Span Deflection (mm) **</th>
<th>% Increasing In Mid Span Deflection *</th>
</tr>
</thead>
<tbody>
<tr>
<td>R – DB – zero</td>
<td>0</td>
<td>300</td>
<td>---</td>
<td>2.9</td>
<td>---</td>
</tr>
<tr>
<td>R – DB – 160</td>
<td>0.2</td>
<td>220</td>
<td>26.7</td>
<td>4.0</td>
<td>37.9</td>
</tr>
<tr>
<td>R – DB – 240</td>
<td>0.6</td>
<td>170</td>
<td>43.3</td>
<td>5.2</td>
<td>79.3</td>
</tr>
<tr>
<td>R – DB – 320</td>
<td>0.8</td>
<td>150</td>
<td>50.0</td>
<td>8.4</td>
<td>189.6</td>
</tr>
</tbody>
</table>

* % = web beam without web opening / web beam with web opening ratio equal to 0.8
** corresponding to ultimate load level of deep beam with opening ratio equal to 0.8

3.4. Stresses in main steel and stirrups bars

Stresses in main steel bars and stirrups have been recorded for each deep beam during the test. Fig. (7) shows the load – stresses in main steel and stirrups bars curves for each tested beam. It can be observed from this figure that the presence of openings in RC deep beam lead to increase the main steel bars and stirrups stresses compared with the control beam. The maximum percentage increasing in main steel bar and stirrups stresses are (63.3 and 875.0) % respectively for beam with opening ratio equal to 0.8.

Figure 7: Load – steel stress curves for all tested deep beams

4. Conclusion

1. Presence of large web opening in RC deep beams caused decreasing in their first cracking and ultimate load capacities and increasing the mid-span deflection. For deep beams with opening ratios 0.4, 0.6 and 0.8, the percentages decreasing in first cracking and ultimate loads were (30.0, 50.0 and 65.0) %
and (26.7, 43.3 and 50) % respectively as compared with beam without web opening. While, the percentages increasing in mid span deflection for the same opening ratios were (37.9, 79.3 and 189.6) %.

2. Presence of large web opening in RC deep beams has affected the main steel bars and stirrups stresses compare with beam without opening. The maximum increasing in main steel bar and stirrups stresses were (63.3 and 875.0) % respectively for beam with opening ratio equal to 0.8.

5. References
Comparison between medical solid waste treatment technologies used in Baghdad hospitals

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² Baghdad university/ college of engineering, Baghdad, Iraq

Keywords: medical solid waste treatment, incineration, steam sterilization technology, advance autoclave device, Baghdad teaching hospitals.

Medical solid waste is considered one of the most important pathogens because it contains the pathological factors that facilitate its transmission directly or indirectly. As these solid wastes pose a serious danger to living organisms and the environment, therefore, the appropriate method of medical solid waste treatment should be chosen, thereby reducing pollution resulting from it as much as possible. This study focuses on two treatment methods used in Iraq which are incineration and steam sterilization technology. Comparison of the two methods showed that advance autoclave device is less hazardous to the environment and society than incinerator.

1. Introduction

Medical solid waste refers to all waste produced within health-care services, medical laboratories and research centers [18]. Waste from hospitals considers special type of toxic and dangerous waste which need an orderly and scientific ways for disposal, so they may be disposed of as economically and hygienically as much as possible to reduce the risk on health and environment. The sequels of contact between waste resulting from hospitals with human are evident and manifold, in addition to that the impact of hospital solid waste on environment and human health outside health care institution cannot be disregarded [9]. Poor management of medical solid waste can cause dangerous diseases to waste workers, patients, staff, and to the public [14-8]. Several problems are resulting from improper medical solid waste management such as environmental pollution in general, unpleasant odors and growth of worms, insects and rodents which considered a factor in the transmission of diseases such cholera, typhoid and hepatitis due to injuries from sharps polluted with human blood [19]. In order to reduce the risks of medical solid waste the first essential step is to segregate waste at their point of generation then provide safe management procedure for the collection, transportation, treatment and disposal of medical solid waste [11]. Numerous medical solid waste treatment methods may be used such as incineration, microwave sanitation, steam sterilization, chemical disinfection and dry heat disinfection [17]. According to medical solid waste treatment studies, approximately 59 - 60% of medical solid waste was treated through incineration while 20 - 37% treated by using steam sterilization, and 4 – 5% treated by other methods [2]. An example on the non-incineration process is a steam sterilization technology, this technology is based on moisture - heat sterilizing where hot steam leads to a change in the nature of the protein of the microbial body which leads to a reduction in infection by pathogenic bacteria [7]. An advantage of steam sterilization technology compared to incineration is the sterilized solid waste considered as non-hazardous waste and can be located in sanitary landfills, whereas ashes resulted from incineration was mostly considered hazardous wastes and must be treated before disposal [15]. The study
aims to compare between two types of medical solid waste treatment technologies used in Iraqi hospitals especially in Baghdad city which are incineration and steam sterilization.

2. Study Area

Three teaching hospitals were chosen in different areas of Baghdad city for evaluating the medical solid wastes treatment device (advance autoclave), these hospitals are:

- Al Yarmouk teaching hospital: A teaching hospital affiliated with the faculty of Medicine /University of Al-Mustansiriyyah and it is a general hospital containing all specialties. This hospital located in the Yarmouk region of Karkh side and its coordinates are (33° 17' 39.5'' N, 44° 21' 9'' E).
- AL Kadhimiya teaching hospital: A teaching hospital located in Karkh side which affiliated with the faculty of medicine /Al-Nahrain University. Its coordinates are (33° 22' 33.9'' N, 44° 19' 35.7'' E) which located in the new Al-Habina where overlooking street 60.
- Al Kindy teaching hospital: A teaching hospital affiliated with the faculty of medicine / University of Baghdad. The hospital located in Sheikh Omar district, Locality of 145 and an alley 13, in Rusafa side and its coordinates are (33° 20' 46.4'' N, 44° 24' 34.8'' E). Table (1) gives a general description of these hospitals.

<table>
<thead>
<tr>
<th>Teaching Hospital</th>
<th>Total Area (m²)</th>
<th>Capacity (no. of beds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al-Yarmouk</td>
<td>26,127</td>
<td>679</td>
</tr>
<tr>
<td>Al- Kadhimiya</td>
<td>21,467</td>
<td>759</td>
</tr>
<tr>
<td>Al- Kindy</td>
<td>48,000</td>
<td>317</td>
</tr>
</tbody>
</table>

Table 1. General description of each hospital

3. Incineration

Incineration is a temperature thermal oxidation process in which organic materials and combustible materials are converted to inorganic and non-combustible materials [4]. This method of treatment take place at high temperatures from 200 °C to more than 1000 °C [18]. Incinerator emissions include sulfur oxides, carbon monoxide, nitrogen oxides, hydrogen chloride, particulate matter, heavy metals volatile organic compounds and dioxins [13]. In addition, the ashes formed from incinerator contain large quantities of very toxic fly ashes which pose significant health risks in long term [6]. A modern conventional incinerator has an equipment for air pollution control, heavy metals and dioxins concentrations were existing in the fine solid which removed from stack gases (fly ashes). The fly ashes were considered as hazardous wastes [10]. The choice of the most suitable control system depends on kind and contaminants concentration. The control systems used in hospital incinerators were typically textile filters, wet scrubbers and dry scrubbers [13]. Although incineration can minimize the weight of solid waste to higher than 70%; after incineration, large amounts of residues "especially bottom ashes" remain, however bottom ashes was described as less contaminated with heavy metals than fly ashes. Previous studies showed that bottom ashes can be used as secondary aggregates in construction materials and roads so, they considered as a valuable resource [12].
3.1 Advantages and Disadvantages of incineration
The incinerators have the following advantages [5]:

1. The mass and volume of solid waste are reduced by 85–90% of its original volume.
2. The reduction of waste is immediate and does not depend on long periods of biological interaction.
3. Incineration facilities can be established near to the municipal solid waste sources or collection points, thus reducing transportation costs.
4. Using the heat recovery technology, the process cost can be recovered from energy sales.
5. To comply with environmental legislative values, air liberalization can be controlled.

Disadvantages of incineration method are [5]:

1. Some materials must not be incinerated as they are non-combustible, or they are valuable for recycling, or their by-products may increase the harmful emissions.
2. The existence of chlorine in municipal solid waste and poor operating practices were a major cause of emissions containing highly toxic furans and dioxins.
3. It may be difficult to control metal emissions that resulting from inorganic waste containing heavy metals like mercury, cadmium, copper, arsenic, lead, etc.
4. Solid waste incinerators require high capital expenditure and trained operators, leading to higher operating costs.
5. Extra fuels are essential to achieve high combustion temperatures.

4. Steam sterilization technology (advance autoclave)
Autoclave is an effective wet thermal sterilization process. It is typically used in hospitals to sterilize reusable medical instruments [16]. In autoclaving, steam or dry heat is introduced into a firmly sealed chamber and medical solid waste inside the chamber must be maintained with temperature range from 121°C to 163°C to destroy spores [2]. Many types of microorganisms are being inactivated if they exposure to sufficient temperature and contact time. The inactivation of microorganisms by this technology expected to reach 99.99%, compared to 99.9999% of inactivation that can be achieved with autoclave sterilization. It should be noted that shredding of medical solid waste must be available before treatment, and milling or crushing can be used for sharps in order to increase disinfection effectiveness, also this type of treatment is unsuitable for anatomical waste, chemical waste and pharmaceutical waste [1].

4.1 The advantages and disadvantages of steam sterilization technology
The advantages of steam sterilization technology are [20]:

1. The investment of disposal facilities construction is low, maintenance and operation are simple, and low operation cost.
2. A convenient processing management, safe and reliable operation, sterilization temperature and pressure can be automatically recorded. The basis for reliability traceability and operation analysis can be provided.
3. 99.9999% of sterilization proportion can be reached and volume reduction reaches to 70%-85%.
4. The disposing quantity is controllable and flexible, and the processing speed rate is quick. Processing equipment can spasmodically "operate according to the mount of disposition", that has good flexibility for medical solid waste quantity fluctuations.

5. The sterilization process does not produce sturdy carcinogenic substances like dioxin. Toxic waste does not generate due to high temperature saturated vapor in the operating medium.

Disadvantages of steam sterilization technology [20]:

1. Steam sterilization process does not appropriate for treatment of pathological waste, drug and chemical waste.

2. A strict classification management for medical solid waste is required because of the limits of treatment object.

3. The steam condensate, and medical solid waste drainage have specific toxicity, which require additional purification treatment.

4. After sterilization, the volume of medical solid waste residue is bigger as compared to incineration technology, so it requires crushing, broken shape, and compression processing.

5. Due to medical waste components, may be possible to produce carcinogenic compound, volatile organic compound, and the mercury vapor which has toxic, disgusting and disagreeable odor.

4.2 Field work on advance autoclave

4.2.1 Advance autoclave components

The basic components of medical solid waste treatment device shown in Fig. 1 are loading belt, shredder’s hopper, sterilization chamber, steam generator, touch screen, printer, vacuum pump, sterile filters and compactor.

Figure 1. Medical Solid Waste Treatment Device
4.2.2 Treatment procedure
Medical solid waste has been placed in bags and then placed on the loading belt, the loading belt convey them to the shredder’s hopper during this time the loading door of the sterilization chamber is opened. The shredder cutting the medical solid waste into small pieces to easy penetrate of the steam. After that, the waste pieces fall into the sterilization chamber by gravity, when the sterilization chamber is completely loaded the shredder stops and loading door is closed, and the sterilization cycle will begin. The operator will choose the specific sterilization cycle from the touch screen of the control panel, the medical solid waste is treated by high pressure saturated steam either of 134 °C or 138 °C cycles. Both cycles have an absolute pressure of 3 to 3.5 bar that can be observed by pressure gauges, and the time of sterilization phase is between 5 to 10 minutes. When the sterilization phase is over, the bottom door of sterilization chamber opens, and sterile waste will fall by gravity and will gather inside trolley. These wastes are then compressed inside the trolley to reduce their volume.

4.2.3 Biological testing
Medical solid waste is treated within the hospital boundary by using advance autoclave or auto-shredder device. This type of treatment is used in all three hospitals. Samples from treated medical solid waste were taken after two days from its treatment and examined in the biology department of Baghdad University to ensure that this treated medical solid waste is free of pathological bacteria and germs. They have been planted on culture media which were blood agar and MacConkey agar, where the MacConkey agar used for the growth of gram negative bacteria while the blood agar used for the growth of both types of bacteria. The samples were planned on these culture media, and have been incubated at 37 °C for 24 - 48 hours in aerobic and anaerobic conditions. The examination showed negative results, which means that this medical solid waste was treated properly and do not have any type of pollutions that may harm human beings or environment.

5. Results and discussion
To know which method is better in terms of treatment of medical solid waste with less hazardous to the environment and society, our findings from the field study about the advance autoclave device were compared with the incinerator based on previous studies on the incinerators. The advance autoclave began to be used recently in some of the hospitals of Iraq, including Baghdad hospitals concerned with the study for medical solid waste treatment. During field study, the method of operation for this device has been identified and examining the results obtained from several samples of medical solid waste treated by it proved that it did not result in any pollution to the environment, where this device works as friend to the environment in that it does not cause any pollution to the air and that the resulting noise was very few so as not to disturb the outer perimeter, in addition, medical solid waste after treatment was harmless and reduced to more than half due to the existence of shredding process before the start of treatment and the compaction of medical solid waste after the end of treatment. Another goal of the shredding process was to increase the efficiency of sterilization as hot steam penetrates into all parts of the medical solid waste. However, although the medical solid waste was well treated and reduced in size, it must be disposed of after treatment by transferring it to the landfill site. Incinerators have been used for a long time as a means of treating medical waste, where medical solid waste was disposed of permanently and only ashes must be disposed of, but the damage caused by incinerator was large, including air pollution because of the gases containing a high percentage of dioxins, furans, and mercury that cause long-term
carcinogenic diseases [3] as well as ashes contain on the proportions of heavy metals that contaminate the soil and groundwater if not disposed of properly. A study made by Ghofran, 2005 [4] in some of Baghdad hospitals proved that misuse of the incinerator may lead to incomplete combustion of wastes and the emission of gases, toxins and contaminants that threaten the environment and the inhabitants, also the ashes resulting from the waste incineration process was polluted and can be considered hazardous wastes due to its environmental impacts on groundwater and soil if it is disposed of incorrectly.

6. Conclusion

It has been proven from the bacteriological tests that the advance autoclave device was effective in the treatment of medical solid waste. The device is environmental friendly where it did not produce air pollutants, and the resulting noise was too low to cause any disturbance to the outside environment. Although incinerator reduce the volume of medical solid waste after treatment to more than 70%, but it produces the flue gases, the fly ashes and slags. It is a major cause of emissions containing highly toxic furans and dioxins which were considered as hazardous gases because they can cause carcinogenic diseases in the long terms. It turns out that the damage caused by the incinerators is much worse than that of the advance autoclave device.

References


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Fatigue life performance of FRP laminates bonded to concrete beams: A parametric investigation

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Keywords: FRP, fatigue life, S-N curve, maximum stress

During the last decades, the number of vehicles crossing bridges has increased worldwide due to population growth. Hence, bridges nowadays experience heavy cyclic loads compared to the initial loads considered in the first steps of design. These heavy cyclic loads produce fatigue degradation and severe damage of the structure if unchecked. One of the most successful techniques used commonly in the upgrading of the structural performance of bridges is the use of fibre reinforced polymer (FRPs) composite materials. The process of strengthening involves adhering the FRP composite to the structural elements using various types of adhesive materials. Although significant research studies conducted to investigate the performance of FRP under monotonic loading, the fatigue life and the associated fatigue damage behaviour are still not fully evaluated. This paper presents a parametric study through finite element analysis (FEA) to predict the fatigue life performance. The FEA model was calibrated using experimental results conducted by the authors. The fatigue life was assessed on the basis of maximum stress and relevant fatigue number of cycles. The outcomes showed that no fatigue issue was observed when the maximum stress ratio is 70% and less from the ultimate static load. Details of FE results and recommendation for future research work are also outlined.

1. Introduction

During the last decades, the number of vehicles crossing bridges has increased in many countries worldwide due to growth population. Hence, bridges nowadays experience heavy cyclic loads compared to the initial loads considered in the first steps of design. In addition, many environmental conditions also effect bridge performance, such as high winds and earthquakes. Fantin [1] pointed out that bridge components require strengthening and upgrading due to lack of flexural strength, inadequate shear capacity, lack of stiffness, and poor durability. One of the most successful techniques used commonly in the upgrading of the structural performance of bridges is the use of fibre reinforced polymer (FRP) composite materials. FRP offers designers an excellent combination of properties, including high strength-to-weight ratio, ease of installation, and corrosion resistance. All these characteristics make FRP ideal for strengthening applications. Rizkalla and Hassan [2] noted that FRP can be applied to defective structural elements as externally bonded or near surface mounted. In recent years, researchers have begun to recognize the importance of investigating the fatigue performance of FRP-to-concrete joints, as the in situ loadings consist of various patterns of static and cyclic loading. Carbon FRP (CFRP) normally has a fatigue resistance better than that of reinforced steel due to the high modulus of elasticity of CFRP, which does not show any noticeable change when tested under tension-tension high cyclic loading [3]. The S-N curve is a well-known tool that use to determine the fatigue life behaviour of materials throughout the relationship between Stress and Number of cycles. The S-N curve is a graphical representation of the number of load cycles required to break a specimen at a range of peak cyclic stress levels. The most common way to represent fatigue data is by means of a Whöler curve, where fatigue strength is plotted...
against the logarithm of the number of cycles to failure. Usually but not necessarily, a linear relation exists between fatigue strength and the logarithm of the number of cycles. In this paper, a parametric study was performed on non-linear finite element analysis to further investigate fatigue life performance by simulation work. The fatigue life behaviour was predicted on the basis of maximum stress ratio versus fatigue number of cycles. This prediction provided sufficient points that enabled the establishment of S-N curve. Details of results and conclusion are outlined in the following sections.

2. Experimental program

2.1. Specimen design
The experimental work included constructing reinforced concrete blocks with dimensions of 400 × 400 × 250 mm. Each concrete specimen was reinforced with cages consisting of N4 Ф12 mm closed ties in each direction and 120 mm spacing between bars. A reinforcement cover of 30 mm was considered.

2.2. Specimen preparation
A sand-blasting was implemented on the larger faces of the concrete blocks 400 × 400 mm. This is to ensure that both surfaces had an adequate level of roughness so that good bond application could be achieved between the FRP material and the concrete surface. After sand-blasting the blocks, the surface was cleaned using a vacuum machine to remove any undesired particles prior to the use in the application of the epoxy material. The bonding area was marked to ensure correct alignment.

2.3. Use of FRP application
The control specimen included a single laminate strip with dimensions of 800 × 120 × 1.4 mm bonded to the surface of the concrete block with a bond length of 370 mm. The FRP application consist of adhering the laminate adhesive over the concrete surface and a CFRP laminate was then bonded to the concrete surface. A custom made steel template was used to ensure a uniform thickness of adhesive can be achieved along the bond line between the FRP and the concrete surface. Fig. 1 shows the procedure of the FRP application used.

2.3. Experimental test setup
The experimental work was conducted on a steel frame first designed by [4]. However, the frame was developed further to comply with fatigue test considerations. The test frame included two 50 mm thick steel plates top and bottom where the specimen was placed in between and secured by 32 mm diameter high tensile strength bar bolted into place. Six strain gauges were attached on the laminate to measure
the strain distribution along the bond line. Fig. 2 shows the location of strain gauges with instrumentation setup of the test frame.

![Test setup and specimen properties](image)

Figure 2. Test setup and specimen properties: (a) test Rig - front view; (b) test Rig – side view; (c) specimen properties and instrumentation.

2.2. Material properties
Most of the material properties supplied by the manufacturers were tested experimentally to verify the properties prior to their use in the FRP application. The concrete had compressive strength of 32MPa with tensile strength of 2.7MPa. The CFRP laminate and the adhesive epoxy had tensile strength of 2906 and 21MPa, and young’s modulus 181, and 10 GPa, respectively.

3. The proposed Finite Element Model (FEM)
The proposed model was implemented in the ATENA 3-D package of non-linear FEA for RC structures [5]. The FEA was used to construct a calibrated numerical model to provide further understanding of the fatigue life performance and mechanism of fatigue damage of FRP laminate bonded to concrete and subjected to different static and cyclic loadings. The calibrated control model was built to represent the actual specimen components. The steel plate of the test frame used in the experimental work was also simulated in this calibration model. Fig. 3 shows the geometry of the control model. The mesh size of micro-elements was assigned particularly over the bond area between the FRP material and the concrete surface where the failure damage is expected to occur. A mesh size of 5mm was assigned across the bonding line. The CFRP laminate and adhesive layers were assigned with mesh size of 1.4 and 1.5mm, respectively. The steel plate of the test frame and the lower part of the concrete block were assigned as coarse mesh size of 20mm to reduce the number of finite elements in the model. Fig. 4 presents the mesh size configuration. The concrete was modelled based on the seared crack model method that used to spread the cracks across the band of elements. The concrete simulated on a non-linear plasticity fracture according to the crack opening law and fracture of energy, where a fixed crack model with constant crack orientation was considered. Based on theoretical modelling, a concrete fracture energy of 80 N/m was
used in this simulation while this value showed satisfactory correlation with the experimental results in relation to failure load and mode of failure. The interface layer of epoxy was modelled to represent the bond behaviour along the joint area between the FRP element and concrete. The parameters of the interface element were collected from the experimental measurements. Research studies such as that of [6] and [7] have proposed theoretical models of an interface element for linear and non-linear fracture mechanisms. In the ATENA 3-D program, the failure criterion of the interface layer was defined from the stress-displacement law, and its micro-elements were assigned to contact volumes in 3-D as per the material prototype (CC3DInterface). The CFRP laminate was modelled as solid elastic material with the same properties used in the experimental work.

4. Results

Static tests were performed first to obtain the average ultimate failure load that used to derive fatigue test matrix based on the maximum stress ratios. The experimental static tests showed that failure occurred with an average of 62.5 kN. In the same line, the FEA showed that the control model failed at 69 kN when subjected to static loading. Overall, the static test results showed good agreement between the experimental measurement and FEM. After this successful correlation was achieved, the FEM was used to predict the fatigue life performance by simulation. A parametric study was implemented with constant minimum stress ratio of 20% and different maximum stress ratios of 85, 80, 75, 70, and 60% of ultimate static capacity. Table 1 summarizes the result outcomes of the parametric study.

<table>
<thead>
<tr>
<th>Fatigue test No.</th>
<th>Stress ratio (%)</th>
<th>Load range (kN)</th>
<th>Average ultimate static failure (kN)</th>
<th>FEM Fatigue number of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>20-85</td>
<td>12.5-53.1</td>
<td>62.5</td>
<td>1,000</td>
</tr>
<tr>
<td>F2</td>
<td>20-80</td>
<td>12.5-50</td>
<td>62.5</td>
<td>40,000</td>
</tr>
<tr>
<td>F3</td>
<td>20-75</td>
<td>12.5-46.9</td>
<td>62.5</td>
<td>1 million</td>
</tr>
<tr>
<td>F4</td>
<td>20-70</td>
<td>12.5-43.7</td>
<td>62.5</td>
<td>188 million</td>
</tr>
<tr>
<td>F5</td>
<td>20-60</td>
<td>12.5-37.5</td>
<td>62.5</td>
<td>&gt;200 million</td>
</tr>
</tbody>
</table>

Table 1. Results of fatigue test predicted by FEM.
The results obtained from the FEA indicate that fatigue damage occurs at fewer fatigue number of cycles, below 1 million cycles, when the maximum stress ratio is 75% and above. However, the fatigue test of the maximum stress ratio of 70% failed after 188 million cycles, indicating good fatigue life performance. Furthermore, the simulation analysis showed that the maximum stress ratio of 60% had an infinite fatigue number of cycles and the fatigue test continued cycling without failure until 200 million cycles. The test was then stopped as an indication of reaching the endurance limit. On the other hand, the parametric study revealed that when the stress range between the minimum and maximum stress ratio is 55% and above, lower fatigue life is obtained. As a result, for safe design considerations, the external CFRP laminate strengthening system will have no fatigue issue when subjected to fatigue loading with a stress range less than 50%. Fig. 5 presents the results of parametric study as monitored by FEA. The figure shows the peak micro-strain recorded for each fatigue test with the corresponding fatigue number of cycles. The cracks initiated from early stage of fatigue loading and propagate progressively along the bond line as the cyclic loading increased. The fatigue test of high maximum stress ratio of 85% shows greater strain measurements in comparison with other fatigue tests. In contrast, the fatigue test with the low maximum stress ratio of 60% had fewer strain readings.

Figure 5. Fatigue test results of parametric study: (a) F1; (b) F2; (c) F3; (d) F4; (e) F5
The outcomes of parametric study were used to develop the S-N curve on the basis of maximum stress ratio and relative fatigue number of cycles, as depicted in Fig. 6. The figure shows the fatigue life performance in accordance with the findings of parametric study. The S-N curve clearly shown that no sign of fatigue degradation was found beyond the maximum stress ratio is 70% of the ultimate static load.

![Figure 6. S-N curve of the parametric study: Maximum stress ratio vs. fatigue number of cycles (log)](image)

**Conclusion**

This paper presented the parametric study to predict the fatigue life performance of CFRP laminate bonded to RC beam and subjected to several cyclic load application. The parametric study performed on ATENA 3-D program of non-linear finite element analysis. The FEA model was calibrated using experimental results conducted by the authors. The study addressed two major fatigue parameters of maximum stress and stress range that are significantly influence the fatigue life performance. These two parameters were used to develop the S-N curve on the basis of maximum stress ratio and relevant fatigue number of cycles. The outcome of this parametric study revealed that external CFRP laminate strengthening system will have no fatigue degradation or debonding issue when subjected to fatigue loading with a stress range less than 50% and maximum stress ratio less than 70% of the ultimate static capacity. However, further research is needed to verify the results, so that design guidelines on the fatigue life behaviour of FRP composites in RC structures could be proposed.

**Acknowledgement**

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References


Creep Strain Development of Self-compacting Portland-Limestone Cement Concrete

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Keywords: creep, high strength concrete, Portland-limestone cement, self-compacting concrete

Prediction of the structural response of reinforced concrete to the time-dependent, creep and shrinkage, volume changes is complex. Creep is usually determined by measuring the change, with time, in the strain of specimens subjected to a constant stress and stored under appropriate conditions. This paper brings into view the development of creep strain for four self-compacting concrete mixes: A40, AL40, B60 and BL60 (where 40 and 60 represent the compressive strength level at 28 days and L indicates to Portland-limestone cement). Specimens were put under sustained load and exposed to controlled conditions in a creep chamber (ASTM C512). The test results showed that normal strength Portland-limestone mixes have yielded lower ultimate creep, meanwhile high strength concrete showed relatively similar creep behavior as ordinary Portland cement SCC but with about 50% lower specific creep.

1. Introduction

In 2009, ASTM allowed the use of up to 5% interground limestone in ordinary Portland cement (OPC) as a part of a change to ASTM C150. Portland limestone cements (PLCs) then were included in ASTM C595. The proposal for increasing the volume of limestone that would be permitted to be interground in cement is designed to enable more sustainable construction, which may significantly reduce the CO₂ that is embodied in the built infrastructure while also extending the life of cement quarries [1]. The phenomenon of gradual increase in strain with time under a sustained stress is called creep. Prediction of the structural response of reinforced concrete to the time-dependent concrete volume changes is complex due to: non-elastic properties of the concrete, continuous redistribution of stress, the effect of cracking on deflection, the effect of external restraints, the effect of the reinforcement, water content and temperature. Failure is controlled by the strains that develop at collapse, while time-dependent strains only affect the structure serviceability. Creep, shrinkage and elastic strains are independent for stresses less than about 40 to 50 percent of the concrete strength, creep strains are assumed to be approximately proportional to the sustained stress, and creep strain can start at stresses as low as 30 to 35 percent of the concrete strength [2]. Many factors affect the creep of concrete such as mix proportions which includes: aggregate quantity, grading and maximum size, water and cement content, air content and admixtures [3] the environment in which concrete is cast and to which it is exposed and loaded during its life has a significant effect on the drying creep of concrete. Drying creep is significantly affected by the relative humidity of the surrounding air. Concrete specimens stored at a constant relative humidity environment of 65% exhibited slightly lower drying creep than concrete specimens stored in an environment that cycled between 40 and 90% relative humidity. The rate of creep at 70 °C (160 °F) for a concrete with a w/c of 0.6, and is approximately 3.5 times higher than that stored at 23 °C (70 °F)[2]. Increasing the load level slightly by 3% or 5% may influence the time-dependent behavior and the predicted critical time, which equals to 96 days and 70 days, respectively, for load levels that correspond to 75% and 77% of the short-term load-carrying capacity [4].
In all concrete structures creep reduces internal stresses due to non-uniform or restrained shrinkage so that there is a reduction in cracking. Creep is assumed to be directly proportional to the applied stress up to about 40% of short time strength [5]. If a concrete specimen is held for a long period under a constant stress, for instance 50 percent of the ultimate strength of plain, reinforced, and prestressed concrete structures. Total creep strain to elastic creep strain, shows that the HS-SCC developed 13% higher creep at 120 days than NS-SCC because HS-SCC has higher paste content [6]. The only statement that can be made with certainty is that creep of silica-fume concrete is not higher than that of concrete of equal strength without silica fume. Limited published data and the different nature of the creep tests used by various investigators make it difficult to draw more specific conclusions on the effect of silica fume on the creep of concrete [7].

2. Materials and Experimental work

Two types of Portland cement were used throughout this study. They were ordinary Portland cement (OPC), confirms to the ASTM C150-Type I [8], and Portland-limestone cement (IL), confirms to the ASTM C595-Type IL [9]. Silica fume (SF) with an activity index of 120% was used as supplementary cementitious material in producing the self-compacting concrete mixes. The silica fume was conforming to the ASTM C1240 [10] and was added by 10% as partial replacement of cement for high strength concrete mixes. Table 1 shows the chemical composition and some physical properties for both types of cement and for silica fume. Limestone dust passing sieve (75 µm) was added to produce SCC, Table 2 shows the properties of the used limestone dust. Natural sand with fineness modulus of 2.97 and SO$_3$ content of 0.3% was used. Natural crushed coarse aggregate with maximum sizes of 10mm and 20mm were used. High performance superplasticizer (Sika Visco-Crete -5930) confirming to ASTM C494 Types G and F [11] was used for producing high strength self- compacted concrete.

<table>
<thead>
<tr>
<th>No.</th>
<th>property</th>
<th>OPC</th>
<th>IL</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Oxide content, %</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>CaO</td>
<td>60.8</td>
<td>61.3</td>
<td>&lt; 1</td>
</tr>
<tr>
<td></td>
<td>SiO$_2$</td>
<td>19.9</td>
<td>17.7</td>
<td>&gt; 85</td>
</tr>
<tr>
<td></td>
<td>Al$_2$O$_3$</td>
<td>4.7</td>
<td>4.1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Fe$_2$O$_3$</td>
<td>3</td>
<td>4.7</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>MgO</td>
<td>1.5</td>
<td>2.8</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>K$_2$O</td>
<td>2.3</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Na$_2$O</td>
<td>0.46</td>
<td>0.1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>SO$_3$</td>
<td>0.11</td>
<td>2.3</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>2</td>
<td>Loss on Ignition (L.O.I)</td>
<td>2.7</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Fineness (Blaine m$^2$/kg)</td>
<td>309</td>
<td>368</td>
<td>&gt; 15000</td>
</tr>
<tr>
<td>4</td>
<td>Specific Gravity</td>
<td>3.15</td>
<td>3.07</td>
<td>2.3</td>
</tr>
<tr>
<td>5</td>
<td>Compressive Strength, MPa at 28 days</td>
<td>42</td>
<td>48.4</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Chemical and physical properties of used ordinary and limestone Portland cement and silica fume.

<table>
<thead>
<tr>
<th>Oxide</th>
<th>SiO$_2$</th>
<th>Fe$_2$O$_3$</th>
<th>Al$_2$O$_3$</th>
<th>CaO</th>
<th>MgO</th>
<th>SO$_3$</th>
<th>L.O.I</th>
<th>CO$_3$</th>
<th>CaCO$_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Content, %</td>
<td>0.21</td>
<td>3.36</td>
<td>0.03</td>
<td>50.84</td>
<td>2.99</td>
<td>0.08</td>
<td>42.2</td>
<td>0.19</td>
<td>98</td>
</tr>
</tbody>
</table>

Table 2. Chemical composition of limestone dust
Four SCC mixes, AI40 and BI60 for Type I cement and AIL40 and BIL60 for Type II cement, as shown in Table 3, were prepared according to the recommendations of ACI Committee 237 [12]. Fresh concrete mixes were tested for self-compactability. Mixes AI40 and AIL40, were designed to yield average compressive strength of 40 MPa at 28 days, with w/c ratio of 0.38 and an aggregate maximum size of 20 mm, the attained strength values were 44 and 41 MPa respectively. Mixes BI60 and BIL60 were high strength concrete. They were designed to yield 60 MPa as average required compressive strength at 28 days and they attained 67 and 62 MPa respectively. These mixes were cast with a water/binder ratio of 0.31 and a maximum size of aggregate of 10mm. For creep test, cylinders of diameter d=100mm and height h=200mm, were exposed to controlled conditions, 21 °C and 35 % relative humidity. Specimens were tested after 7 days of water curing; they were arranged in the creep device according to the ASTM C512/C-512M-15 [13] as shown in Fig.1. After that a load of 0.4 f ’c was applied on the tested specimens and remained sustained for 240 days. At the same, drying shrinkage strain (ASTM C157) was measured under the same controlled conditions for another specimens cast from the same mixes.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Binder, kg/m³</th>
<th>Limestone Dust, kg/m³</th>
<th>Aggregate, kg/m³</th>
<th>Max Size of Agg, mm</th>
<th>Water, kg/m³</th>
<th>ViscoCrete L/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>AI40</td>
<td>400 – I</td>
<td>0</td>
<td>100</td>
<td>764</td>
<td>20</td>
<td>152</td>
</tr>
<tr>
<td>AIL40</td>
<td>400 – II</td>
<td>0</td>
<td>100</td>
<td>800</td>
<td>20</td>
<td>152</td>
</tr>
<tr>
<td>BI60</td>
<td>450 – I</td>
<td>50</td>
<td>50</td>
<td></td>
<td>10</td>
<td>155</td>
</tr>
<tr>
<td>BIL60</td>
<td>450 - II</td>
<td>50</td>
<td>50</td>
<td></td>
<td>10</td>
<td>155</td>
</tr>
</tbody>
</table>

Table 3. Mix details

3. Results and Discussion
3.1 Properties of Fresh SCC:
Results of the self-compactability tests for the studied SCC mixes are listed in Table 4. The results revealed that the produced mixes were all conforming to the requirements of the international EFNARC [14]. The replacement of Portland cements with silica fume has enhanced the passing and filling abilities of the tested mixes. This behavior could be attributed to the high surface area of the silica fume.
### 3.2 Creep Strain Development

Fig. 2 shows the creep behavior with time for the four mixes, it indicates low creep strain when using limestone cement. For high strength concrete mixes, the creep strain was lower than normal strength concrete, due to the low w/c ratio, small aggregate maximum size, presence of silica fume which has chemical and physical effect and the high modulus of elasticity. Mixes AI40, AIL40, BI60 yielded higher creep than mix BIL60 by 26, 16 and 4 %, respectively.

<table>
<thead>
<tr>
<th>Type of mixes</th>
<th>Slump flow</th>
<th>V-funnel</th>
<th>L-Box</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>T&lt;sub&gt;500&lt;/sub&gt;</td>
<td>Sec.</td>
</tr>
<tr>
<td>AI40</td>
<td>750</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>AIL40</td>
<td>750</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>BI60</td>
<td>790</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>BIL60</td>
<td>790</td>
<td>2</td>
<td>6</td>
</tr>
</tbody>
</table>

**Table 4. Fresh SCC test results**

![Creep strain results for all mixes](image)

At the moment of loading, a strain occurs and is termed as initial elastic strain. It depends on the duration of load application and it may be determined by using standardized procedures for the experimental determination of static elastic modulus (referring to the short interval strain after the application of loading) as tabulated in Table 5.

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Actual Stress (0.4 f’c), MPa</th>
<th>E, GPa</th>
<th>Elastic Strain, microstrain</th>
</tr>
</thead>
<tbody>
<tr>
<td>AI40</td>
<td>17.6</td>
<td>30</td>
<td>587</td>
</tr>
<tr>
<td>AIL40</td>
<td>16.4</td>
<td>29</td>
<td>565</td>
</tr>
<tr>
<td>BI60</td>
<td>26.8</td>
<td>38</td>
<td>705</td>
</tr>
<tr>
<td>BIL60</td>
<td>24.8</td>
<td>37</td>
<td>670</td>
</tr>
</tbody>
</table>

**Table 5. Initial elastic strain and elastic modulus at 28 days**

It can be noticed from Fig. 3 that when the initial strain was added, the four mixes showed the same behavior with different levels of response, and the limestone cement BIL60 with high strength still gave the slightly lowest creep than other mixes AI40, AIL40, BI60 by 9%, 4% and 4%, respectively.
Creep coefficient is defined as the ratio between creep strains to the initial strain [3] see Fig.4. Both Portland-limestone and ordinary Portland concrete mixes have nearly the same creep coefficient for the same strength level. These coefficients are 2.763, 2.761, 2.212 and 2.223 for AI40, AIL40, BI60 and BIL60, respectively. On the other hand, high strength concrete mixes have lower creep coefficient than normal strength concrete.
The specific creep, which is defined as creep strain per unit stress, for the four mixes is displayed in Fig. 5. Portland-limestone mixes give slightly higher specific creep than ordinary cement concrete, which are AIL40=60.7, BIL60= 33, while AI40=59 and BI60=31.9. High strength concrete mixes give approximately half specific creep of the normal strength concrete.

4. Conclusions

Based on the present work, the following conclusions could be made:

1. For normal strength concrete, Portland-limestone cement gives lower creep strain compared to ordinary Portland cement due to lower clinker content which is responsible for the creep of concrete. On the other hand, increasing the strength level of concrete made Portland-limestone mixes to behave just like ordinary Portland cement ones with respect to creep development.
2. When the initial elastic creep strain is added to the recorded creep, all mixes have almost the same behavior.
3. High strength concrete has about 50% lower specific creep than normal strength concrete.

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[7] ACI Committee 234, Guide for the Use of Silica Fume in Concrete- ACI 234R-06, American Concrete Institute, USA (2013)


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Longitudinal Strain Readings Using Different Measurement Techniques

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Keywords: Strain measurements, steel behaviour, strain gauges, photogrammetry.

Structural engineering theories and design developments were built upon strain measurements for decades. Being able to accurately measure strain on a structural member is very crucial to ensure better quality and longer life span for most engineering structures. Several ways to measure strain are available and being used in many researches. The most common methods used are either experimental such as image correlation photogrammetry and strain gauges or could be analytical such as finite element modelling. However, each of the mentioned methods have its own associated limitations related to time consuming and intense specimen preparation. A scant studies were made to investigate different photogrammetry pattern layouts to advise the best and fastest layout techniques. Therefore, this paper presents a comprehensive comparison between different photogrammetry pattern layout techniques as compared to fundamental strain measuring techniques which are strain gauges and finite element modelling. Nine Dog-Bone shaped steel specimens were tested through three different strain measurement configurations. The results showed that using spray painted patterns or roller stamped patterns in photogrammetry is as accurate and reliable as the manual dotting which is the advised method. However, they are much faster to establish on the tested elements and does not waste time that could be invested in other parts of the work.

1. Introduction

Strain is essential in determining stresses of materials while they are being loaded, this is a requirement that will ensure better quality and longer life span for most engineering structures. For example, the measurements of steel members’ strength could be either obtained by means of stress required to force the material to undergo a plastic deformation or maximum stress that the member can withstand, which both require the ability to capture strain [1]. In addition, measuring strain is of a great deal of importance when dealing with structural components in order to evaluate their strength and service life [2]. Most strain measuring methods are based on stress-strain relationship or stress-strain diagrams.

Several studies have identified different strain measuring techniques. Each technique has its associated advantages and limitations regarding the accuracy of the results. In the meanwhile, there are three main techniques that have been an active search area in the last decades. Two of them are experimental which are strain gauges and image correlation photogrammetry, while the other one is theoretical that is finite element analysis.

Latest researches and articles have presented different strain measuring techniques, but experimentally the use of image correlation photogrammetry was the best approach for different engineering practices. The main focus of this paper will be on filling the gap in the available literature specifically related to patterns problems that take time to be established in image correlation photogrammetry. The research is also aiming to investigate new layout techniques to establish these patterns in a way that is faster and at the same time as accurate as traditional ways of measuring strain. The main outcome of this research will
be a comprehensive comparison of strain measurement results obtained from three different tests to investigate if the new method of pattern layout is valid or not. The strain measurement in this research will include the use of traditional strain gauge, image correlation photogrammetry (using traditional (manually dotted) pattern layout) and image correlation photogrammetry (enhanced pattern layouts). In addition, a non-linear finite element 3D model will also be created to measure the strain of a replica of the test specimens and will be used as a basis to compare the obtained results from image correlation photogrammetry against. Strain gauges are electrical devices that can measure strain at a specified location [3]. A strain gauge consists of metal foils or wires (depending on the gauge type) that are arranged in a pattern that allows it to elongate until it reaches its elastic limit. According to Smith (2005), strain gauges were developed in the 1950s by Lord Kelvin who first observed the electrical resistance in metal cords. Therefore, this observation later paved the way for the invention of strain gauges. After that Arthur Ruge and Edward Simmons in the early 1930s discovered that strain gauges can measure strain [3]. From that point, different types of strain gauges have been developed and widely used in several applications such as aircraft manufacturing and other engineering aspects. Strain gauges mainly operate depending on the elongation or shortening of the patterns of the gauge. From the change in electrical resistance, it can easily measure the surface strain accurately. However, it also has some limitations as they cannot work as an independent unit and it needs to calibrate to produce results. Image correlation photogrammetry or Digital Image Correlation DIC is a simple, flexible and high precision system that provides full-filed displacements and strain measurements [4, 5]. The history of photogrammetry was developed in the late 1480s by Leonardo da Vinci, who established the very first principles of geometric analysis of images [6, 7]. After many inventions in the field of photogrammetry, the idea remained a concept until the invention of photographs by Daguerre in 1837 [6, 8]. Consequently, many developments in the field of photogrammetry were established. Image correlation Photogrammetry mainly works by capturing digital images and tracing the patterns that is applied to the specimen and then comparing the specimen surface before and after the deformation to obtain the required displacements and strain readings as shown in Fig.1. It is used in different fields such as geology, biology, space and many other fields. Although Image correlation photogrammetry is a very advantageous and important technology, it has some limitations such as specimen preparation and sensitivity of the device.

![Figure 1. Correlating images before and after the deformation [9].](image)

2. Methodology

In this research, a total of 9 Dog-Bone shaped steel specimens were used and tested through three different strain measurement techniques. The specimens were statically loaded using 250MTS machine with a displacement rate of 2mm/min up to yielding and the rate is increased to 5mm/min. Vic-3D system and analysis software were used to capture the strain in in addition to strain gauges. The rate at which the data were captured in both ways was 2Hz. The main outcome of these tests was to obtain strain readings
that can be analysed and used further in a comprehensive comparison between the three configurations. The three configurations are similar in terms of strain gauges (one on each side of the specimen), YFLA-5-1L Single element strain gauges (High elongation strain gauges) with CN Cyanoacrylate adhesive were used. This specific type of gauges was used in order to achieve 12-17% of strain readings beyond yielding to create a good base for the comparison. However, three different pattern layout techniques were used to establish the photogrammetry patterns which are spray painted pattern, manual dotting and finally VIC Speckle Pattern Application Kit pattern. All the tested specimens had the same steel properties and undergone the similar preparation work except for how the photogrammetry pattern were established.

2.1. Specimen Design
The steel specimens are designed in a dog-bone shape which is the common practice in tensile testing. The dimensions of the steel dog-bone were designed in accordance with AS1391 standard as shown in Fig. 2. However, this standard has some restrictions regarding the design of the specimens that this research did not follow as it is more interested in capturing strain rather than steel properties. Based on that few alterations were made to increase the width of the effective part of the dog bone to make capturing strain and preparations of the specimen a lot easier. Hot rolled steel with a yield stress of 350Mpa was used to form the specimens. However, the grade of the steel was a little different than what the manufacturer claimed it to be.

![Specimen dimensions](image)

**Figure 2.** Specimen dimensions.

2.2. Strain Gauges Application
To achieve the best results, the specimens initially need to be cleaned and sand-blasted or scraped by using course and fine sand paper. By using a pen (rolled tip pen that is made of tungsten as it is capable of scratching the steel) and a ruler, the centre of the specimen should be identified and straight lines should be drawn. The centre of the specimen is expected to be the location of the failure and this is where the strain gauges will be applied. Then, acetone and cotton wipes is used to clean the area in order to achieve better grip for the strain gauge. Note that the specimen should not be left more than 30 minutes in this state otherwise rust will reappear and the process should be repeated. Using the duct tape, the gauge is carefully placed on the specified centre of the specimen in the adhesive is added beneath the strain gauge. After that the strain is left to set on the specimen for about five minutes. Strain gauges in this stage ready
and all it is needed is to connect the strain gauge to the machine by attaching the wires of the strain gauge to the system. But before doing so, the strain gauges needs to be tested using a multi-meter to check the usability of the gauges.

2.3. Photogrammetry Speckle Pattern Application
The image correlation photogrammetry technique is dependent on a contrasting pattern on the tested specimen surface. This pattern can be either applied manually in most steel applications or naturally occurring for some concrete applications. The purpose of this report is to advise the fastest and best ways of applying the speckle patterns. Therefore, three main patterns will be tested, that is dotting, spraying and roller stamps as shown in Fig.3. These patterns will be applied after applying the gages and painting one side of the specimens with white paint. There will be three specimens for each pattern that should be tested.

2.4. Alias Problem
Alias is basically a problem that appears when the pattern is too small and the camera resolution may not be enough to capture this pattern. The pattern should be scaled correctly to the specimen; it should not be too small as alias will occur and not too large as the subset may be entirely on black or white area and there will be many other similar spots on the specimen. In an aliasing case, the subset does not always move accordingly with the specimen and the applied pattern will show a jitter and it will interact with the sensor pixels and therefore, it will result in what is called a Moiré pattern [17]. However, while testing the sprayed specimens in this report, alias appeared as shown in Fig.4, but the problem was solved as there is an option in the system which minimise aliasing.
3. Results and Discussion

This part of the paper will present the data collected during testing after they have been analysed. Besides, it will present a comprehensive presentation of each method of strain capturing. In addition, it will explain the data analysis procedure step by step in order to reach high understanding of the presented results. Based on the data analysed and the comparison, the conclusion will be formed and presented in the final chapter.

3.1. Strain Gauge and Photogrammetry Analysis

Fig. 6 shows that strain gauges achieved the required strain readings percentage which is nearly 17% after yielding. The photogrammetry data extracted from the three different points on the specimen showed similar readings to the strain gauges. From these graphs, it is obvious that all photogrammetry pattern layout techniques were successfully able to collect accurate strain readings as compared to the strain gauges. That means that these patterns are reliable and accurate and can be trusted to capture strain on structural elements during testing.

![Figure 6. Photogrammetry and Strain Gauges results.](image)

4. Conclusion and Recommendation

The required tests were conducted to test different pattern layout techniques and compare them to strain gauges and finite element model results. Three pattern layouts were tested with three specimens for each technique which are spray paint, roller stamps and dotting. After a comprehensive analysis of the results obtained, it was shown that all the layout techniques achieved reliable strain measurements as compared with each other and with the standard methods. That shows that using any of these techniques will guarantee reliable strain measurements. However, the dotting technique was the most time consuming and requires intensive specimen preparation. Based on that and the results, it is highly recommended to use either the spray paint or the roller stamps if applicable. These methods proved their efficiency in terms of the accuracy of the results.
5. Acknowledgment
This project would have been impossible without the help of the brilliant staff in Swinburne Smart Structures Lab. A special thanks to Kevin Nievaart, who helped with the testing procedures, without his guidance and effort the tests would have never been accomplished.

References
MECHANICAL ENGINEERING

Synthesis and Mechanical Properties of Al-based Metal Matrix Composite Reinforced with Milled Carbon Fibres via Powder Metallurgy: Microstructures and Wear properties

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Keywords: Metal Matrix Composites, Uniball milling, Al-MCFs, Hot Pressing, Mechanical properties

Aluminium matrix reinforced with different volume fractions of milled carbon fibres (MCFs) were manufactured via advanced powder metallurgy. Precursor composite powders containing 5 to 20 vol. % of MCFs were prepared by 50 hours of milling using the Uniball mill, whereas the monolithic composites were produced by uniaxial hot pressing (HP) at 600 ± 10 °C for 15 minutes in an argon atmosphere. Characterization was performed using the X-ray diffractometry, field emission scanning electron microscopy (FSEM), and abrasive wear testing. Results show that Uniball milling reduces the aspect ratio of the MCFs, refines the Al grain size, and produce a uniform distribution of MCFs when using 15 and 20 vol. % of MCFs. The reinforcement with MCFs improves the wear resistance by acting as lubricant during the abrasion of two sliding surfaces. The combination of Uniball milling technique and uniaxial hot pressing can facilitate the development of Al-MCFs composites to use in automobile industries.

1. Introduction

Aluminium matrix composites (AMCs) are considered as advanced materials possess high strength and stiffness, low coefficient of thermal expansion, high thermal and electrical conductivity, and good wear resistance. Their applications include sport, aerospace, defence and automotive fields. Aluminium and its alloys were broadly used in metal matrix composites (MMCs) due to lower cost of manufacturing with various processing routes and high strength to weight ratio. Carbon fibres (CFs) are valuable reinforcement for AMCs because of their low density, high strength, and commercial availability (as continuous and chopped fibres) [1–6].

Two primary processing methods of AMCs are liquid phase processing and solid-state processing or powder metallurgy (PM) [3]. Al/CFs composite has been manufactured using PM and stir-casting method [6], thixomixing and ultrasonic liquid infiltration [7,8]. The main problems encountered in producing Al/CFs composite include; poor reactivity and wettability of CFs by molten Al, poor processability, and low interfacial properties [5,6]. The field of manufacturing of Al/CFs using the milling techniques is still under research [3].
In this paper, precursor composite powders of Al reinforced with different amounts of milled carbon fibres (MCFs) were prepared by dedicated milling and mixing technique described as Uniball magneto-milling [9,10]. These blends and refined powders were then uniaxially hot pressed (UHP) at 600 °C in an argon atmosphere to produce monolithic composites. Precursor powders and products were investigated by X-ray diffractometry, field emission scanning electron microscopy (FSEM) equipped with energy dispersive spectroscopy (EDS), and pin on drum wear testing.

2. Experimental

Properties of the starting materials are shown in Table 1. Mechanical milling was performed by magnetically controlled Uniball mill under low energy ball-particle shearing mode [11,12]. The milling parameters were 65RPM milling velocity, 300kPa Argon atmosphere, 27/1 ball to powder mass ratio (PBR), and four chromium steel ball of 25 mm diameter as the milling media. The Al powder was reinforced with different volume fraction 5, 10, 15 and 20 % of MCFs and the blends were milled for 50 hours. Monolithic composite samples were produced by uniaxial hot pressing (UHP) at temperature 600 ± 10 °C, uniaxial pressure 70 ± 5 MPa, and 15 minutes total processing time. Metallographic preparation for characterisation was carried out by cutting, mounting, grinding and polishing using: Struers Accutom, Struers CitoPress-30, and Struers Tegramin systems respectively. Then Leica EM RES101 ion milling system was used to clean the polished surface of any residual debris. The Archimedes densities were measured according to the ASTM B962-15 [13] using 4-digits balance while the theoretical densities were calculated using the rule of a mixture [3]. Microstructures were investigated using JEOL-JSM-7001F field emission gun scanning electron microscopy (FSEM) equipped with energy dispersive X-ray spectroscopy (EDS) silicon drift detector using 15 kV voltage and 10 mm working distance.

Pin on drum abrasive wear test was carried out following ASTM G 132-96 [14] with the selected parameters: 6 ± 0.5 mm diameter, applied load 20, 40, and 60 N, translation speed 0.04 m/s, rotational speed 8.9 rpm, and sliding distance 6.02 m. The counter drum surface was covered with 150 grit garnet paper as abrasive surface and the test was carried out at 20 °C and 50 % humidity. The wear rate was calculated from the equation $W= \frac{\Delta m}{\rho L F}$, where: $W$ is the wear rate in mm$^3$/mm N , $\Delta m$ is the mass loss in g, $\rho$ is the density in g/mm$^3$, $L$ is the sliding distance in m, and $F$ is the applied load in N [14]. The worn surfaces of the composite samples were examined by scanning electron microscope (JEOL JSM-6490LV).

<table>
<thead>
<tr>
<th>Materials</th>
<th>Al</th>
<th>Milled carbon fibres</th>
<th>Stearic acid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Matrix</td>
<td>Reinforcement</td>
<td>Process control agent</td>
</tr>
<tr>
<td>Density</td>
<td>2.7 [g/cm$^3$]</td>
<td>1.8 [g/cm$^3$]</td>
<td>0.847 [g/cm$^3$]</td>
</tr>
<tr>
<td>Particle size</td>
<td>Fine powder</td>
<td>$L=100$ [μm], $Dia.=7.5$ [μm]</td>
<td></td>
</tr>
<tr>
<td>Hardness</td>
<td>0.22 [GPa]</td>
<td>2.9 [GPa]</td>
<td>Powder</td>
</tr>
<tr>
<td>Melting point</td>
<td>660 [°C]</td>
<td>3500 [°C]</td>
<td>361 [°C] boiling point</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>Sigma-Aldrich</td>
<td>Easy Composites Ltd</td>
<td>Sigma-Aldrich</td>
</tr>
<tr>
<td>Purity</td>
<td>≥ 91.9 %</td>
<td>100 %</td>
<td>99.99 %</td>
</tr>
</tbody>
</table>

Table 1. The properties of starting materials used in this work.
3. Results and discussion

3.1 Starting materials and X-ray diffraction

Figure 1 shows the XRD patterns (a and b) and the SE micrographs (c and d) of starting Al powder and MCFs, in which the Al particle was irregular in shape with sharp edges (Figure 2c), while the MCFs (Figure 2d) have rod shape with 7.5 μm in diameter and 100 μm long. Figure 2 shows the XRD patterns of Al-MCFs powder composites over different volume fractions of MCFs to measure the Al peak shifting and broadening and investigate any evolved phases after 50 hours milling time. In which an increasing the volume fraction of MCFs up to 20% lead to increase the intensity and peak shifting to the right. The XRD approved that no other phases formed after milling and the MCFs peaks were disappeared in the XRD due to the high scattering background and lower volume fraction. Figure 3 shows the crystallite size and lattice strain of the composite powders as a function of the volume fraction of MCFs where the crystallite size and lattice strain were both increased with increasing the MCFs volume fraction up to 20 %vol. MCFs.

Figure 1. (a & b) show the XRD patterns of starting Al powder and MCFs respectively, while (c & d) show the SE micrographs of starting Al powder and MCFs respectively.

Figure 2. XRD patterns of Al+20%MCFs as a function of volume fraction after 50 hours.
3.2 Density and Microstructures observations

Table 2 shows the Archimedes, theoretical, and relative densities of Al-MCFs composites, in which the density decreased as the volume fraction of MCFs increases due to the lower density of MCFs. The relative densities appear more than 100% which indicates dense compacts and may also indicate inter-diffusion, high solute atom concentrations, excellent adhesion, and precipitates at the interface. Another interpretation of the extra number may be because of impurities that come from the milling operations.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Archimedes density g/cm³</th>
<th>Theoretical density g/cm³</th>
<th>Relative density %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al+5 % vol. CF</td>
<td>2.6824 ± 0.006</td>
<td>2.6450</td>
<td>101.4135</td>
</tr>
<tr>
<td>Al+10 % vol. CF</td>
<td>2.6550 ± 0.004</td>
<td>2.5900</td>
<td>102.5111</td>
</tr>
<tr>
<td>Al+15 % vol. CF</td>
<td>2.5731 ± 0.006</td>
<td>2.5350</td>
<td>101.5028</td>
</tr>
<tr>
<td>Al+20 % vol. CF</td>
<td>2.5700 ± 0.008</td>
<td>2.4800</td>
<td>103.8957</td>
</tr>
</tbody>
</table>

Table 2. Archimedes and theoretical densities of hot pressed Al-MCFs composites.

Figure 4 shows the SE micrographs of hot pressed Al-MCFs composites, in which the black is the MCFs, the white spots is the milling contaminations milling equipment (ball and vials), and the grey is the Al matrix. Uniform distributions of MCFs appeared at 15 and 20% of MCFs with approximately 70% of MCFs were broken during milling. The iron impurities were pushed close to the surfaces of MCFs after hot consolidation.

Figure 5 shows the secondary electron and backscattered micrographs with energy dispersive X-ray spectroscopy (EDS map) of hot pressed Al+20% vol. MCFs in which the MCFs homogenously distributed.
along the Al matrix with lower agglomerates. These images reveal that no third phase (i.e. $\text{Al}_4\text{C}_3$) were recognised at the interphase between Al and carbon fibres. The reaction between Al and C appeared at the interface at temperatures above 500 °C to form $\text{Al}_4\text{C}_3$ [15,16]. Oxygen was evaluated by less than 2% which is coming from Al, MCFs, polishing, and vacuum. The mechanical milling helps to break down the oxide layer on the surfaces of the Al particles and MCFs which enhances wettability at a relatively elevated temperature of uniaxial hot pressing. These images also show the random distribution of MCFs along the Al matrix in different orientations 90°, 180°, 45°, and 60°, which help to improve the composite strength and increase the load bearing in various directions.

![Micrographs and energy dispersive spectroscopy (EDS) of Al+20 % vol. MCFs composite sample after UHP.](image)

**Figure 5.** SE and backscattered micrographs and energy dispersive spectroscopy (EDS) of Al+20 % vol. MCFs composite sample after UHP.

### 3.3 Wear testing results

Figure 6 a and b shows the volume loss and specific wear rates as a function of applied load after pin on drum abrasive wear testing, in which the volume loss and wear rate decreases as the volume fraction of MCFs increases while Figure 6d shows the secondary electron and backscattered images of the worn surfaces of Al reinforced with 10 and 20 vol. % MCFs. The MCFs play as a lubricant in abrasive wear that helps to reduce friction between the two sliding surfaces. The composite reinforced with 10 % MCFs has a higher volume loss than 20 % vol. MCFs. Wear rate increases as the applied load increases showing straight line relationship. The different surface morphology amongst different volume fraction of MCFs is distinguished. The combination of self-lubricating reinforcements, such as carbon fibres, to produce the composites help to improve the wear resistance of Al matrix [17,18].
4. Conclusions

Precursor powders of Al-MCFs were prepared by Uniball magneto milling and then uniaxially hot pressed into a dense monolithic product with lower contamination. MCFs were distributed uniformly along the Al matrix with no evidence of Al4C3 phase. The wear results indicate a decrease of volume loss and wear rates as the volume fraction of MCFs increases which improves the wear mechanism because the carbon debris help to prevent the counterpart components from wearing in the tow body sliding system. Finally, the optimised manufacturing parameters were found as 50 hours of milling followed by uniaxial hot pressing at 600°C for 15 minutes.

5. Acknowledgement

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References


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Design and Simulation study of Effective Cooling Channel for Injection Moulded Plastic Part

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Keywords: cooling channel, conformal cooling, injection, shrinkage, warpage

Injection moulding (IM) process has significant impact on plastic manufacturing process. In IM process cycle, cooling time carries almost half of its cycles time where cooling channel performance is one of the most crucial factors because it has significant effect on both production rate and the quality of the plastic part. With the advent of additive manufacturing, one of the effective way to address this issue is by manufacturing mould with conformal cooling which can control the uniform temperature distribution during plastic solidification process. This paper presents design and simulation study of conformal cooling channels and compare with conventional cooling for an industrial plastic part. Comparative study includes ejection temperature, shrinkage, temperature profile, and part warpage to determine which configuration is more appropriate to provide uniform cooling with minimum cycle time. Autodesk Simulation Moldflow Insight (ASMI) software is used to examine the results of the cooling channels performance.

1. Introduction

The plastic materials are used widely in our daily life involving many fields in term of building, furniture, aviation, automation, and aerospace, their importance came from the distinctive properties such as, corrosion resistance, lightness, and easy to be fabricated [1]. Injection mould process is the most common method to produce the plastic parts for economic mass production and product quality [2]. The injection mould involves melting the plastic material and inject it inside the cavity of the mould and then the molten material will be cooled down in order to be ejected, thus the main phases in injection moulding process are injection, packing, cooling and ejection [3].

The desired products undergo many important factors and issues which could reduce the final quality and production rate, such as, cycle time, warpage, and shrinkage. Time to reach ejection temperature part (or time to freeze), represent the time taken from injection to demoulding temperature. Moreover, it explains the filling and packing phases when the hot plastic enters to mould cavity and also effect on the cooling time. Warpage is one of the main defects in an injection moulded plastic parts, which can be described as a distortion and causes bending/twisting of the part. Warpage generally caused by the uneven cooling which results non-uniform shrinkage, as well as, depends on the part/mould design; however, there are other reasons; for instance, gate location, runner type, and cooling system [1]. On the other hand, shrinkage in the plastic parts has many causes such as, residual stress during filling/packing/cooling and packing pressure, which has an inverse relationship with shrinkage [1]. In order to solve the previous issues there are two main solutions, first of all, redesign the mould, and secondly, find the best process parameters (mold temp./melt point/cooling system/packing pressure) [4]. This paper aims to determine the best cooling system configuration known as conformal cooling.
which provides uniform cooling, minimum cycle time, less warpage and shrinkage. Three different cooling systems configurations have been designed and investigated, which are automatic cooling channel, conventional cooling channel, and conformal cooling channel.

2. Methodology

2.1. Part Modelling (CAD)

Parametric Technology Corporation (PTC) Creo parametric 0.3 software has been used to create Computer Aided Design (CAD) model of the plastic bowl. The dimensions of the bowl (Fig. 1) without handle are height 96 mm, diameter 200 mm. CAD model then exported to Initial Graphics Exchange Specification (IGES) surface model to be imported in ASMI software. (Fig 1).

![Figure 1. CAD model of bowl](image1)

2.2. Autodesk Simulation Moldflow Insight

ASMI software is a comprehensive plastic flow simulation software which assists the researchers/plastic engineers to specify the perfect integration of the plastic part shape/materials/mould design and the different processes parameters, which ultimately simulate high-quality products [5]. In this study, the IGES model of plastic CAD model has been imported as a dual domain mesh model because the thickness of plastic bowl is 2 mm which does not require 3D meshing.

2.2.1. Meshing

The initial global edge length of the mesh sizes has been refined to 7 mm from 12.43 mm. The results in mesh statistics show that there are many warning messages errors related to mesh elements intersections. So, 5 mm global edge length has been chosen to generate the mesh and the merge tolerance is 0.10 mm, the number of elements are 28682 triangular elements, and zero element intersection, free edge and 88.7% match percentage (Figure 2 A). The best injection location has been chosen on the bowl bottom according to the gate location analysis, and the cold tapered sprue with L100, D8, d4 mm dimensions has been created at the best location (Figure 2 B). Moreover, some mesh checking has been done to ensure if there is any problem with meshing connectivity between the part and the cold sprue using mesh repair wizard, (Figure 2 C).
2.2.2. Material selection

Bowl is made with Polypropylenes (PP) material, trade name is PPT 1070 and the manufacturer name is Kemcor Australia. Properties of the plastic part are shown in Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Family name</td>
<td>POLYPROPYLEN</td>
</tr>
<tr>
<td>Trade name</td>
<td>PPT 1070</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>Kemcor Australia</td>
</tr>
<tr>
<td>Melt temperature range</td>
<td>260°C</td>
</tr>
<tr>
<td>Absolute maximum temperature</td>
<td>300°C</td>
</tr>
<tr>
<td>Ejection temperature</td>
<td>153°C</td>
</tr>
<tr>
<td>Material structure</td>
<td>PP</td>
</tr>
<tr>
<td>Material ID</td>
<td>54272</td>
</tr>
<tr>
<td>Grade code</td>
<td>KC 104</td>
</tr>
<tr>
<td>Supplier code</td>
<td>KEMCORAU</td>
</tr>
</tbody>
</table>

Table 1. Properties of plastic part

2.2.3. Processing Conditions

Setting accurate process parameters using ASMI can be considered as close as to real condition which allows reporting all the process parameters of mould and part. Table shows all process parameters that have been used for this study.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mold surface temp.</td>
<td>40°C</td>
</tr>
<tr>
<td>Mold open time</td>
<td>5 sec.</td>
</tr>
<tr>
<td>(Injection + packing + cooling) time</td>
<td>30 sec.</td>
</tr>
<tr>
<td>Ejection temperature</td>
<td>153°C</td>
</tr>
<tr>
<td>Max. part frozen percentage at ejection temp.</td>
<td>100%</td>
</tr>
<tr>
<td>Mold temperature convergence tolerance</td>
<td>0.10000</td>
</tr>
<tr>
<td>Min. number of mold temperature iterations</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 2. Processing conditions

2.3. Cooling Channels

The appropriate design of cooling channels system should be considered because of their significant effect on the products. Where, it can reduces both cooling time and defects, and increases both the productivity and quality [2] & [6]. During designing the cooling channels there are some important factors such as connection of channels, coolant compositions and pressure drop in term of coolant and runners [7]. Generally, most of cooling channel systems created by using the straight drilling and plug unsuitable holes, however, by using new technology like additive manufacturing it can create more complex cooling channels which provide high cooling conditions to extract the heat quickly.
2.3.1. Surface area of cooling channel

The surface area of the cooling channel has a significant effect on the cooling rate [6]. Where, the surface area for all types of cooling channels should be equal or within 5% different between them. Therefore, comparative analysis will be more meaningful to investigate the best cooling channel systems. Cooling channels diameters are 10 mm and the coolant liquid is pure water with 10 °C and Reynold number\(^1\) is 11476.7. Three types of cooling channels will be compared, automatic (ACC) with length 6760 mm, conventional with length 6900 mm and conformal cooling channels with 6757 mm.

- Surface area of automatic cooling = \((2\pi r h) + (2\pi r^2) = (2\pi \times 5 \times 6760) + (2\pi \times 25) = 212421\) mm\(^2\)
- Surface area of conventional cooling = \((2\pi r h) + (2\pi r^2) = (2\pi \times 5 \times 6900) + (2\pi \times 25) = 216817\) mm\(^2\)
- Surface area of conformal cooling = \((2\pi r h) + (2\pi r^2) = (2\pi \times 5 \times 6757) + (2\pi \times 25) = 212327\) mm\(^2\)

2.3.2. Automatic cooling channels (NCC)

This kind of cooling channels can be created manually or automatically on the XY axes, and it can be created by using Cooling Circuit order (Figure 3 A) and use the following data:

1. Channel diameter = 10 mm
2. How far the cooling channel from the part in both sides = 25 mm.
3. Number of channels = 8 mm, Distance between Channel Centres = 30 mm
4. Distance to extend beyond part = 25 mm

2.3.3. Conventional Straight Cooling Channel (CSCC)

CSCC is the most common cooling system because of its simplicity to fabricate using traditional straight drilling. However, these types can’t follow the part profile, especially circular parts, thus the efficiency of cooling will be affected. CSCC has been created by using nodes and straight lines in PTC Creo parameter and then exported as an IGES file extension with surface model and Datum curves and points. In ASMI software, all nodes and lines defined as channels with 10 mm diameter, 0.05 mm roughness, and 10 °C coolant temperature, (Figure 3 B).

2.3.4. Conformal Cooling Channel (CCC)

CCC represent the best cooling system because they follow the part surface profile. However, the manufacturing of these types is very difficult by using the straight drilling. Thus, they need special technology to be create such as additive manufacturing. In our part, the conformal cooling channels have been created by using curves in PTC Creo parameter and then exported as an IGES file extension with surface model and Datum curves and points. In ASMI, these curves defined as cooling channel with 10 mm diameter, 0.05 mm roughness, and 10 °C coolant, (Figure 3 C).

\(^1\) Reynold number \((R_e = \frac{\rho V D}{\mu})\), where, \(\rho = 1000\) kg/m\(^3\), \(V = 1.5\) m/s, \(D = 10\) mm, \(\mu @ 20^oC = 1.004\) m\(^2\)/s
3. Results and discussion

For three types of cooling channels, time to reach ejection/demoulding temperature, volumetric shrinkage, and warpage deflection have been investigated and shown in Table 3.

<table>
<thead>
<tr>
<th>Type</th>
<th>NCC</th>
<th>CCC1</th>
<th>CCC2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to reach ejection temperature, part (sec.)</td>
<td>23.65</td>
<td>20.67</td>
<td>18.10</td>
</tr>
<tr>
<td>Volumetric shrinkage (%)</td>
<td>16.73</td>
<td>16.69</td>
<td>16.62</td>
</tr>
<tr>
<td>Warpage (mm)</td>
<td>2.77</td>
<td>2.83</td>
<td>2.85</td>
</tr>
</tbody>
</table>

Table 3. Analysis results for time to reach ejection (sec.), volume shrinkage (%), and warpage (mm)

3.1. Time to reach ejection temperature part

The simulation results for time to reach ejection temperature part, shows that for automatic cooling channel is 23.65 sec, while is less using conventional cooling channels which is 20.67 sec as shown in (Figure 4 A, B). However, it decreases more to 18.10 sec using conformal cooling channels as can be seen in (Figure 4 C). The main reason is because automatic and conventional cooling channel configurations does not maintain uniform cooling because of uneven distance from the plastic surfaces [8]. In contrast, following the part geometry in the mould has been obtained by conformal cooling configuration system, which shows the lowest value of time to ejection temperature.

3.2. Volumetric shrinkage

The volumetric shrinkage has been analysed, and the results of the analysis show that the conformal cooling channel has the less shrinkage value 16.62 %. Whereas, the conventional cooling channels is a bit more volumetric shrinkage value 16.69 % compared to the conformal cooling system and stills the best than automatic cooling channels value 16.73 % as shown in (Figure 5 A, B, C). Conformal cooling
channels has good results because the good distribution of cooling between the part and the mould, which came from the design of the conformal channels by following the surface profile.

![Figure 5. A, B, C volumetric shrinkage using ACC, CSCC, CCC](image)

### 3.3. Warpage analysis

In terms of warpage analysis, the results of the warpage analysis for three cooling channels types have very close values compared with the original part without cooling channels which is 2.92 mm. In automatic cooling channels is 2.77%. However, the values in conventional cooling channels and conformal cooling channels were 2.83 mm and 2.85 mm respectively as can be seen in (Figure 6).

![Figure 6. A, B, C, D warpage analysis using ACC, CSCC, CCC](image)

### 4. Conclusion

Three different cooling channels have been compared in term of its efficiency by using the Autodesk Simulation Moldflow Insight software. Efficiency parameters are taken for these studies are time to reach ejection temperature or demoulding temperature, volumetric shrinkage, warpage deflection. The results show that the conformal cooling channels displayed significant improvement among the other cooling configuration because these types of cooling channels follow the part profile or conform with the part surface which is in contact with mould cavity surfaces, thus provides uniform heat extraction from the molten plastic by conduction heat transfer. However, the conformal cooling channels need special kind of fabrication technique, additive manufacturing provides these facilities and it can create very complex cooling channel where conventional straight drilling cannot create such cooling configuration.
References


Numerical simulation of the compressive behavior of functionally graded lattice structures

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Keywords: Lattice structure, functionally graded, finite element analysis, deformation behaviour.

Lattice structures have attracted attention of a large number of applications such as personal protective equipment and packaging due to their distinctive properties, in particular combining the lightweight and high strength. Previous work on the mechanical properties and energy absorption capability of lattice structures has been experimentally investigated. In this research study, finite element models have been developed using LS-DYNA code of ANSYS® software to investigate the compressive properties and energy absorption capability of cubic lattice structures. The investigated lattice structures were uniform and functionally graded lattice structures with corresponding relative density. Both uniform lattice and functionally graded lattice models were meshed using 3D solid element and subjected to quasi-static compressive loads. Finite element analysis results were found to agree well with previous empirical findings of the functionally graded lattice structure. The functionally graded lattice structures exhibited distinctive deformation behavior than uniform one, where by the collapse of layers starts sequentially, starting with a lower density layer to higher density one in sequence. In contrast, uniformly dense lattice structures were homogeneously deformed under compressive loads. The results also showed that the energy absorption behavior was distinct and increased with increased compressive loads in the functionally graded lattice. These results increase the potential that the functionally graded lattice structures or other models of density gradient would be more desirable for an application that required high energy absorption capability.

1. Introduction

Lattice structures or cellular structures have been widely used in various applications such as personal protective equipment and packaging, structural lightweight, thermal insulation, energy absorption and bio-medical implant due to their unique properties. Cellular structures have received intensive studies to investigate their mechanical and physical properties in last few decades. The most important research work is that performed by Gibson and Ashby[1]. They have conducted a comprehensive study on investigating the deformation behaviour and mechanical properties of the different cellular models such as honeycomb, metallic foam and natural cellular structures with a range of volume fraction. In general, different manufacturing methods that have been traditionally used to produce the metallic lattice structures, for example melt gas injection has been used to form a metallic foam and then machine it to the desirable shape[1]. For the same purpose, investment casting [2], stacking and joining the laminar plates in a periodic manner [3, 4], physical vapour deposition (PVD) have been employed. However, these conventional fabrication methods have some limitations. For example, they are costly and unable to produce lattice structures with a complex shape for advanced uses [5]. Their effectiveness to the
shape complexity [6] and discrepancy in properties of fabricated cellular structures are also some of the main limitations.

Recently, additive manufacturing technology opens new window to manufacture the lattice structures for a wide range of applications due to its ability to overcome the limitations of the conventional techniques [5, 7-9]. Several attempts have been experimentally conducted to study and investigate the mechanical compressive behavior and energy absorption capability of cellular structures made of additive manufacturing [5, 10-14]. The drawbacks associated with additive manufacturing processes are high cost and slow fabrication process, but these are expected to get better with continued development in this technology [9, 15].

Accordingly, conducting the numerical simulation, which can predict the mechanical properties of lattice structures, can decrease the required experimental works as well as the manufacturing cost. In addition, finite element analysis is able to simulate and verify the experimental work, which increases the recognition of the deformation response and mechanical properties of a new designing approach of the lattice structures due to the ability to identify the stress distribution and high stress concentration regions before the fabrication step. Smith et al. [13], Lee et al. [16], Zargarian et al. [9] and Zhong et al. [17] developed the finite analysis models to investigate the mechanical properties, and to verify the experimental results of different types of uniform lattice structure.

In this study, the mechanical properties and energy absorption capability of uniform as well as functionally graded lattice structures are investigated numerically. No previous works have been reported on investigations the mechanical characteristics and energy absorption property of the functionally graded lattice structures with continuous and smooth density change using finite element analysis. Most researches have focused on an experimental study in investigations the functionally graded lattice structures with abrupt, step-wise and pore size density changes in every layer [5, 8, 10]. A quasi-static compression test was simulated to study the deformation behaviour under the compressive loads of both types of lattice structures numerically using LS-DYNA export code of ANSYS© software.

2. Methodology

2.1 Design of the lattice structure

The investigated lattice structures in this study measuring (30×30×30 mm) were designed using PTC Creo™ Parametric ©3.0 software. The lattice structures were built of F2BCC unit cell. The F2BCC lattice unit cell consists of 12 solid struts with circular cross-section by which they intersected at 45° angle to vertical, four at the cell centre, and eight at the four faces of the cell (two struts at each face) as shown in Fig 1 (a). It was duplicated in three directions (X, Y, Z) in order to build whole lattice structure, thus providing lattice structures contain six layers. In this study, investigation the mechanical performance and energy absorption capability of functionally graded lattice structures and assess and compare with uniform lattice structures based on identical relative density is the main purpose. The relative density was assigned to both lattice structures 0.185. In meeting this value, functionally graded lattice structures were designed with density increase continuously gradually from top to bottom throughout the structure layers at 35% constant rate. This was achieved by design cell struts diameters varying from (0.38 - 1.113 mm) for the structure. The cell struts diameters were changed in one direction with linear and continuous change, thus resulting in smooth density change at the boundaries of the lattice structure layers. Fig 1 (a) shows the CAD model and schematic of the unit cell, and (b) CAD model of the
designed functionally graded and uniform lattice structures. In lattice structures with uniform density, this was achieved through the design of uniform cell struts with 0.7 mm diameter.

**Figure 1.** CAD model and schematic of unit cell (a), CAD model of uniform (left) and (b) graded lattice (right)

### 2.2 Finite element analysis of mechanical properties

To simulate the compressive behaviour of the lattice structures, LS-DYNA code of ANSYS© software was used to generate the model and solve the equations of finite element analysis. For both lattice structures uniform and graded, 3D solid elements were employed to mesh lattice models with six degrees of freedom. During the numerical analysis, the FE models were assigned with material properties based on Alomarah’s study [18] that shown in table 1, which is supposed to be a bilinear material response under compression tests. Since the lattice structures have separated small areas (nodes) on the top and bottom of the models Fig 1 (b), which may cause some errors during the applying boundary conditions, two connected plates were modelled on top and the bottom to overcome this issue. The boundary conditions were applied to simulate exactly what had happened in experimental test [5, 8, 10], in which the top plate was freely moved in Z direction at the constant velocity and fixed in other directions, and the bottom one was fully fixed in all degrees of freedom Fig 2. In addition, the type of contact between the rigid plates and the model was used an automatic node to surface contact, while the automatic single surface contact was used to define the interaction between the connected solid struts of the cells [7]. For clarification, no boundary constraints were used to the sides of the lattice models during the all simulations.

<table>
<thead>
<tr>
<th>properties</th>
<th>Density (g m⁻³)</th>
<th>Young’s modulus (GPa)</th>
<th>Poisson ratio</th>
<th>Yield strength (MPa)</th>
<th>Tangent modulus (GPa)</th>
<th>Ultimate strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AISI-12</td>
<td>2.7</td>
<td>15</td>
<td>0.33</td>
<td>165</td>
<td>3.3</td>
<td>220</td>
</tr>
</tbody>
</table>

**Table 1.** Properties of aluminium alloy material [18]

**Fig 2.** Boundary conditions of FEA models of compression test for (a) uniform, (b) graded lattice
3. Result and discussion

3.1 Deformation behavior of FEA models

Figure 4 shows the resulting data of the compressive stress-strain curves of FEA models. The predicted deformation behavior of FEA models under the compressive loads for both uniform and graded density models presents in Fig 3 (a, b) respectively. Unfortunately, the running simulation for whole structure could not be conducted due to facing some of the constraints. Computational time required and capability of available computers are the main constraints that we had faced during the finite element analysis. To overcome these issues, and by considering the symmetrical geometry of the designed lattice structure, one tower of the structure contains six layers with one cell in each layer was considered to conduct the numerical analysis. It is found that the deformation response of FEA model of uniform lattice found to be discrepancy with experiment collapse behavior [5, 8, 12]. The deformation process of FEA - uniform model was deformed in a corresponding manner and homogeneous as shown in Fig 3 (a), in which starts with buckling and bending in solid struts of the cell and is followed by full compaction due to contact of the cell struts with the next cell struts. The difference is due to the difficulty in simulating the mechanism of collapse, which is a combination of 45° shear, fracture and flake formation in some struts at the same time.

Compared to the FEA- uniform density model, the FEA model with gradual density exhibited close agreement with the empirical results of the previous work [5, 8, 10]. Fig 3 (b) elucidates the collapse behavior of FEA – graded model. It can be observed that the deformation process is entirely different from the collapse behavior of FEA - uniform model. The deformation behavior of FEA – functionally graded model exhibit a distinctive deformation behaviour by which commenced with low density cell at the top, and then, in sequence, the collapse of cell-after-cell continuously. However, it is worth mentioning that the gradually dense lattice structure exhibited distinct collapse behavior under the compressive loads than uniform lattice structures. These findings increase and support the potentials that the functionally graded lattice structures are more desirable for applications with high impact resistance [5, 8, 10].

![Figure 3](image_url)

**Figure 3.** Predicted deformation behaviour of (a) uniform, and (b) graded lattice structure under compressive loads.
3.2 Energy absorption response of FEA models

The determined values of energy absorption of both FEA models provide in Table 2. The energy absorption capability for both FEA-uniform and FEA-graded lattice structures has been determined as the area under the compressive stress - strain curve using the numerical integration, which is represented by equation (1).

\[ W_v = \int_0^\varepsilon \sigma(\varepsilon) d\varepsilon \]  

The cumulative energy absorption per unit volume versus the increase of compressive strain for FEA-uniform and FEA-graded lattice structures are plotted in Fig 5.

<table>
<thead>
<tr>
<th>Energy absorption (MJ/m³)</th>
<th>FEA models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniform</td>
</tr>
<tr>
<td>5.06 ± 0.008</td>
<td>6.27 ± 0.006</td>
</tr>
</tbody>
</table>

Table 2. Energy absorption properties of FEA models

It is clear that the FEA-graded lattice structure was able to absorb energy higher than the FEA-uniform lattice structures, in which 6.27 MJ/m³ and 5.06 MJ/m³ are the values respectively. It has been observed that the energy absorption response of FEA-uniform lattice structure increases at a constant rate and in an almost linear relationship with the compression strain values. This is due to the homogeneous deformation of the uniform lattice under the compressive loads as shown in Fig 4. On the other hand, the energy absorption behaviour of the FEA-graded lattice structure was quite different as shown in Fig 5.

The graded lattice structure exhibits distinctive behaviour of energy absorption due to its deformation response under compressive loads as explained in the previous section. The gradual increase of absorbed energy was tied with the deformation behaviour of layer by layer starting with the top layer of low density one. These results and observations are found to agree with reported values of energy absorption experimentally from the literature that supports the functionally graded lattice structures [5, 8, 10].

![Compressive stress-strain](image)

**Fig 4.** Compressive stress-strain of FEA models

![Cumulative energy absorption](image)

**Fig 5.** Cumulative energy absorption per unit volume against compressive strain curves of FEA models
4. Conclusion

It was demonstrated that numerical simulation of the compressive behaviour and energy absorption capability of uniform and functionally graded lattice structures can be performed successfully using LS-DYNA code of ANSYS software. The finite element analysis has proved that the functionally graded lattice structure exhibits distinctive characteristics in deformation under compressive loads. The collapse process of graded structure is a non-homogeneous layer by layer crushing beginning with the collapse of the lower density layer and then to higher density one in sequence, while the uniform lattice structure exhibited homogenous collapse in the whole structure. The energy absorption behaviour of the graded lattice structure increased continuously and gradually with increase in the strain value while the uniform lattice structure shows closer to a linear relationship. These results indicate that functionally graded lattice structures would be more attractive for applications that required high impact or shock resistance. Further investigations are needed on more advanced density gradient profile for various applications. In addition, combined finite element analysis and experimental study are recommended for optimising the design of new density gradient structures.

5. References


The Experimental Investigation of Butanol-Diesel Blend on Engine Performance and Emission Levels in DI Diesel Engine

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Keywords: butanol; diesel engine performance; exhaust gas emissions.

Diesel engines are widely used for commercial vehicles due to their high efficiency. However, oxides of nitrogen (NOx), carbon monoxide (CO) and unburned hydrocarbon (UHC) emissions of diesel engine cause a negative impact on public health and environment. Using butanol as additive for diesel is one way to reduce diesel engines emissions. This paper investigates diesel engine performance of 10% and 20% butanol (B) blended with diesel fuel. Engine performance such as brake power (BP), brake-specific fuel consumption (BSFC) and in-cylinder pressure were measured. Exhaust gas emissions such as NOx, CO and UHC were also assessed and compared with conventional diesel as baseline. The test was carried out in a single-cylinder direct injection (DI) diesel engine at three engine speeds (1400, 2000 and 2600 rpm) at full load. The experimental results revealed that: BP of butanol-diesel blends was comparable with neat diesel at low and medium engine speeds, while the peak in-cylinder pressure was higher compared to diesel at medium engine speed. Exhaust gas temperature (EGT) and NOx emissions were reduced as a result of the addition of butanol to the diesel blend, while UHC emissions were slightly increased. CO emissions were significantly reduced due to butanol-diesel blend has high oxygen content.

1. Introduction

Diesel engines are widely used for commercial vehicles due to their high efficiency. However, oxides of nitrogen (NOx), carbon monoxide (CO) emissions of diesel engines cause a negative impact on public health and environment. In addition, the growing number of alternative biofuels [1, 2] such as butanol has led to increase and accelerate interest in studying combustion characteristics of these fuels in diesel engine. Using butanol as additive for diesel fuel is one way to reduce diesel engines’ emissions. Butanol fuel has advantageous due to its comparable properties to conventional diesel fuel. Some of these advantages are shown: (1) less corrosive behaviour on fuel injection systems because the hygroscopic nature is lower; (2) higher flash point which means it is a safer option for storage and distribution; (3) lower vapour pressure, thus producing less evaporative emissions [3]; (4) production manner from crop waste, reducing dependence on fossil fuel [4] (5) benefits of emission reduction such as (soot, smoke, NOx, CO) due to high oxygen content which resulted in complete the combustion and reduced temperature of combustion [5-7] and (6) higher laminar flame speed of butanol resulted in enhance reactions [8]. The aims of this study is to investigate the impact of butanol-diesel blend on direct injection (DI) diesel engine performance and emission levels at three engine speeds (1400, 2000 and 2600 rpm) at full load.
2. Fuel Preparation and Properties

Normal butanol (B) was used at 99.8% analytical grade and were obtained from Chem-Supply Australia. Diesel was obtained from a local Toowoomba (Australia) petrol station as a baseline. 10% and 20% butanol (B) were blended with neat diesel, referred to B10D90 and B20D80. The density was measured using volumetric and weighting measurement for all test fuel blends at 20 ºC room temperature. The kinematic viscosity at 40ºC of the test fuel blends was measured using a Brookfield viscometer. The heating values of the blends were measured using a digital oxygen bomb calorimeter in the University of Southern Queensland lab. Each test was carried out in triplicate. Table 1 shows separate fuel properties. Figure 1 shows important fuel properties of test fuels.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Diesel (D)</th>
<th>Butanol (B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/L)</td>
<td>0.82-0.86</td>
<td>0.810</td>
</tr>
<tr>
<td>Viscosity (mm²/s) at 40 ºC</td>
<td>1.9-4.1</td>
<td>2.235</td>
</tr>
<tr>
<td>Surface tension (mN /m)</td>
<td>23.8</td>
<td>24.2</td>
</tr>
<tr>
<td>Calorific value (MJ/kg)</td>
<td>42.65</td>
<td>33.1</td>
</tr>
<tr>
<td>Cetane number</td>
<td>48</td>
<td>17-25</td>
</tr>
<tr>
<td>Latent heat of vaporisation at 25 ºC (MJ/kg)</td>
<td>270</td>
<td>582</td>
</tr>
</tbody>
</table>

Table 1 shows the properties of the separate fuel [1, 2].

Figure 1. Important fuel properties of test fuels.

3. Experimental Setup and Procedure

3.1. Engine Test Setup

The engine test was conducted using a single-cylinder, four stroke, water-cooled, DI diesel engine. An electrical dynamometer connected to the engine was used to control the load. The crank angles were measured using a crank angle encoder setup on the shaft of the engine. A Kittler 6052C pressure transducer (CT400.17) and charge amplifier connected to a data acquisition system with (electronic indicating system) (CT 400.09) were used to record cylinder pressure values at one crank angle revolution for 50 cycles each test. The exhaust gas emissions were analysed using a Coda gas analyser to measure NOx, CO, CO2 and UHC. The accuracy ranges of the Coda gas analysers are 0.1% CO2, 10 ppm NOx, 1 ppm HC and 0.01% CO. The accuracy ranges of the other equipment are: thermocouple in the exhaust manifold 0.0001°C; pressure transducer in cylinder head, 0.000001 bar; fuel flow, 0.0001 l/h; tachometer on the output shaft, 0.0001rpm and load cell on dynamometer 0.0001 Nm. Figure 2 shows schematic diagram of test set-up. Table 2 contains the engine specifications.
3.2. Engine Test Condition
The engine was heated up until steady-state conditions were reached. The experiments started with testing neat diesel (D100), then B10D90 and B20D80 blend were tested. When new fuel was tested, the fuel system was emptied and cleaned with a vacuum pressure pump to remove all the old fuel that was in the injection system. In addition, after that, the engine was filled with new fuel and operated at least 20 minutes to get a sufficiently stable operating condition then recording commenced. The engine tests were conducted at engine speeds of 1400, 2000 and 2600 rpm and at the same compression ratio 19:1 in all experiments with full load. The experiments were carried out in triplicate to reduce the experimental error.

3.3. Heat Release Rate (HRR) Formulation
The heat release rate was calculated from the cylinder pressure data and crank angle readings that were recorded in the data acquisition of each test. The analysis was derived from the first law of thermodynamics for a closed system with an ideal gas after the compression stroke before the exhaust valve opens.

\[
\frac{dQ_n}{d\theta} = \frac{\gamma}{\gamma-1} P \frac{dV}{d\theta} + \frac{1}{\gamma-1} V \frac{dP}{d\theta} = \left( \frac{1}{\gamma-1} \times P_i \times (V_{i+1} - V_i) \right) + \frac{1}{\gamma-1} \times V_i \times \left( P_{i+1} - P_i \right) / \Delta \theta
\]

Here: \(dQ_n/d\theta\) is the heat release rate per crank angle (CAD), \(\gamma\) (ratio of specific heat) = 1.35 is the ratio of specific heats. The input values are the pressure data (collected as function of crank angle) and the cylinder volume \(V\) at any crank angle \(\theta\) (calculated from the engine geometry) [9].

---

Table 2. Engine specifications.

<table>
<thead>
<tr>
<th>Engine model</th>
<th>G.U.N.T. Hamburg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combustion type</td>
<td>Direct Injection (DI) Engine</td>
</tr>
<tr>
<td>Number of cylinders</td>
<td>1</td>
</tr>
<tr>
<td>Compression ratio</td>
<td>5:1-19:1</td>
</tr>
<tr>
<td>Bore</td>
<td>90mm</td>
</tr>
<tr>
<td>Stroke</td>
<td>74mm</td>
</tr>
<tr>
<td>Capacity</td>
<td>470cm³</td>
</tr>
<tr>
<td>Nozzle injection pressure</td>
<td>300 bar</td>
</tr>
</tbody>
</table>
4. Results of Engine Test and Discussion

4.1. Combustion Characteristics

4.4.1. In-Cylinder Pressure and Heat Release Rate (HRR)

Figure 3 and Figure 4 present the relationship between the peak in-cylinder pressure trace and HRR and the crank angle of the test fuels at 1400 and 2000 rpm. B20D80 blend gives a maximum peak in-cylinder pressure compared to neat diesel at 2000 rpm due to the low cetane number (CN) of the butanol-diesel blend (Table 1). The maximum peak in-cylinder pressure increased with butanol addition as a result of enhancing air-fuel mixing caused by the long ignition delay time of the alcohols [10, 11]. This result has an agreement with result reported in Ref. [12]. HRR of both butanol blends was lower compared to neat diesel at both engine speeds (Figure 4).

![Figure 3. In-cylinder pressure at engine speeds 1400 rpm and 2000 rpm.](image)

![Figure 5. BP and BSFC of test fuels at three engine speeds.](image)

4.3. Emissions Characteristics

4.3.1. Exhaust Gas Temperature (EGT) and NOx Formation

Figure 6 presents the EGT and NOx emissions of the test fuels at various engine speeds. All butanol-diesel blends showed slightly reduction in both EGT and NOx emissions at all engine speeds. The maximum reduction in NOx and EGT was found by 6.9%; 10. 39%; 4.69% and 6.2% respectively for B10D80 and B20D80 blends at 1400 rpm. This trend could relate to: the high oxygen content and the lower cetane number of the butanol-diesel blends. These complications led to delays in ignition time and resulted in decrease in the premixed zone. This process can reduce the local temperature and result in decrease NOx emissions. The result has an agreement with results found in Refs. [13, 14].
4.3.2. Carbon Monoxide (CO) and Carbon Dioxide (CO2).

CO emissions can be formed via a number of mechanisms. A rich mixture will lead to increase CO emission. Figure 7 shows CO and CO2 emission levels for the different ratios of test fuel blend at three engine speeds. The figure shows a noticeable reduction in CO emission value of test fuel blends compared with neat diesel due to complete combustion consequences of high oxygen content of butanol. The maximum reduction in CO emission was found by 54.7% and 60% of B10D90 and B20D80 at high engine speed. The higher oxygen content of butanol-diesel fuel blends can promote the oxidation of CO and enhance the complete combustion, resulting in lower CO emissions. The figure shows also that CO2 emission was correlated with brake power (BP) at almost all engine speeds and butanol ratios.

4.3.3. Unburnt Hydrocarbon (UHC)

The use of butanol-diesel blends increased the UHC emissions compared to neat diesel at low and high engine speeds (Figure 8). It is clearly noticed that butanol leads to poorer fuel-air mixing and insufficient time to complete the reaction due to the higher heat of evaporation of the butanol blends (582 MJ/kg) compared than that neat diesel (270 MJ/kg) and high spray penetration value of butanol blend caused unwanted fuel impingement on the chamber walls resulted in increased UHC emissions. The maximum increment in UHC was found by 11.3% and 6.4% of B10D90 and B20D80 respectively at low and high engine speed. This result has an agreement with finding in Refs. [6, 15, 16].
5. Conclusions

This experiments revealed varying results of test fuel blends. Engine test experiments were carried out in a single-cylinder DI diesel engine. The combustion characteristics, engine performance and exhaust gas emissions of test fuel blends were assessed and compared. Some conclusions follow:

- BP of butanol-diesel blends was comparable with diesel, while the peak in-cylinder pressure was higher compared to neat diesel at medium engine speed.
- EGT and NOx emission of B10D90 and B20D80 blends were reduced by 6.9%, 10.39%, 4.69% and 6.2% respectively at 1400 engine speed as a result of the addition of butanol to the conventional diesel, while UHC emissions were increased by 6.4% and 11.3%.
- CO emissions were significantly reduced by 54.7%-60% of B10D90 and B20D80 blend at high engine speed due to butanol-diesel blend has high oxygen content resulted in complete the combustion.

6. References


Factors affecting the application of knowledge in the IT service desk function in the higher education sector in Australia

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Keywords: Information Technology Service Management (ITSM), Service Desk function, Knowledge Management, Knowledge Application, Employee-Driven Innovation Enablers.

Information Technology Service Management (ITSM) is a discipline that deals with the management of IT services. One important function in ITSM is service desk (SD) which is a part of the technical support that keeps services running. SD uses knowledge systems (explicate knowledge) in their activities. When no systematic knowledge is found, SD relies on their personal knowledge (tacit knowledge). Prior studies examined several aspects of knowledge management (KM) in ITSM, such as knowledge creation and sharing. However, mere creation and sharing of knowledge do not necessarily create value or improve the performance of ITSM. According to knowledge-based theories, performance depends on knowledge application capability effectiveness (KACE) rather than on knowledge itself. Individuals have several reasons to create and access knowledge but not apply it. Causes are not clear to date, but include lack of opportunity and time, risk and distrust of the source of knowledge. KACE is needed to improve the SD (e.g. reduce time of handling incidents). This study attempts to understand how SD applies the knowledge that is made available. This study proposes using employee-driven innovation enablers (EDIE) (time-outs, Expansion roles, competitions and open forums) as the key drivers for KACE. To this end, the study employed a qualitative approach (interviews) to elicit input from 23 SD managers working in Higher Education Sector in Australia (HESA). The results of the interviews are reported in this article.

Keywords: Information Technology Service Management (ITSM), Service Desk function, Knowledge Management, Knowledge Application, Employee-Driven Innovation Enablers.

1. Introduction

For the purpose of this study, following Kim and Lee (2010), KACE is defined as a process of applying (i.e. act upon it) available knowledge for the purpose of solving problems and dealing with challenges. Previous research has contributed greatly to our understanding of knowledge creation (KC) and knowledge sharing (KS) (Conger & Probst, 2014; Graupner, Basu, & Singhal, 2009; Liang & Baozhang, 2009; Nabiohlahi, Alias, & Sahibuddin, 2011). However, KACE as an important factor (Alavi & Leidner, 2001; Alavi & Tiwana, 2002; Grant & Baden-Fuller, 2004; Nickerson & Zenger, 2004) is still neglected
(Shahsavaran & Ji, 2011). Although some of the prior attempts in the literature have identified the importance of KACE in improving the performance and creating value of knowledge resources, these attempts have not focused on factors that could affect this aspect (Davenport, De Long, & Beers, 1998; Trusson, Doherty, & Hislop, 2014). This study attempts to address this issue through qualitative methodology, using interviews conducted with IT managers in selected universities in Australia. This paper first provides a review of literature on both KM and IT SD, which is followed by a theoretical framework underpinning the research gap. Then, data collection method and analysis are given. Next section presents the findings and discussions. The paper ends with conclusions and future research directions.

1.1. Literature review

In recent years, there has been an immense focus on ITSM research area by different researchers. The majority of ITSM research is driven by IS scholars (Shahsavaran & Ji, 2011). Efforts in this area aim to increase the body of knowledge, as well as help practitioners to adapt or implement better IT service initiatives. Shahsavaran and Ji (2011) reported that of 152 studies published between 2000 and 2010 in journals and conference proceedings, only seven studies have addressed aspects of KM issues in ITSM. The main focus of the seven studies is on knowledge creation (KC) and knowledge sharing (KS). For instance, Graupner, Basu, and Singhal (2009) aimed to design a domain wiki for better ITIL knowledge-sharing. Another example is M Jantti and Eerola (2006) who discussed using KM in ITSM for Problem Management (PM). The main aim of their work was to evaluate the usefulness of the IT service PM model as well as to present a theory-based model for PM. In their research, M Jantti and Eerola (2006) found that there is no knowledge base available for the SD, which causes difficulties. In fact, nowadays knowledge is available for SD workers in myriad forms (e.g. SKMS, YouTube, personal knowledge), but the utilisation of knowledge, which is defined as knowledge application capability effectiveness (KACE), from different sources is the issue (Pawliczek & Rössler, 2016; Waswas & Kraishan, 2017). In the literature, KACE is the most important part of any KM initiative (Alavi & Tiwana, 2002; Marko Jantti, 2013; Ortiz & Benitez, 2014).

Innovation literature offers some important insights into the underlying factors of KACE in IT service desk function. For example, Gressgård, Amundsen, Aasen, and Hansen (2014) and Birkinshaw and Duke (2013), found employees driven innovation (EDI) that helps or hinder the application of knowledge in organisations. EDI is based on the assumption that innovation can be achieved by employees using their hidden capabilities (Kesting & Parm Ulhøi, 2010). Smith, Ulhøi, and Kesting (2012, p. 72) describe EDI as “deliberate changes to a firm’s bundle of routines or parts thereof that have been “driven” by “ordinary” employees, who have no formal authority to be involved in such decisions”. Work in the area of innovation has demonstrated how EDI can affect KACE of IT employees (Høyrup, Hasse, Bonnafous-Boucher, Møller, & Lotz, 2012). Birkinshaw and Duke (2013) discussion clarified these enablers through the use of four factors: time out, expansive roles, competition, and open forums.

The term time-out in this paper refers to an IT employee having time away or short period during the work in order to rest, plan what to do next. This view is consistent with Teglborg et al. (2012) who claim that an employee can use this time to apply the available knowledge outside their formal role. Davenport et al. (1998), observe that, as far as there is a lack of time provided to employees in order that they may apply their knowledge is concerned, KACE is one of the most negated area. Past studies of innovation
have shown that the use of time out can indeed facilitate the application of knowledge among staff members (Smith et al., 2012; Teglborg et al., 2012; Wihlman, Hoppe, Wihlman, & Sandmark, 2014).

Expansive roles is the status of giving chance for a junior employee deal with a new task instead of a senior one (Høyrup et al., 2012). The idea of expansive roles is based the assumption that employees can be directed to think outside the box (Davis, 1997). In a very profound essay, Rubalcaba, Michel, Sundbo, Brown, and Reynoso (2012) found that expansive roles provide essential opportunities for staff members to practice their knowledge (e.g. find new solution for a problem). This is really important as Wasko and Faraj (2005) argued that the lack of opportunity might be a reason behind poor KACE, however, it appears to be neglected. Several studies indicate that expansive roles can provide opportunities for individuals to use their knowledge more effectively.

Open forums refers to a situation or meeting in which people can talk about problems or matters especially in the workplace. According to Rubalcaba et al. (2012), open forums provide a collaborative learning experience where staff members discuss and debate about how to address problems that emerge. Moreover, past studies have also shown that open forums play a vital role in stimulating knowledge exploiting in organisations (Davis, 1997; Høyrup, 2012). In addition, Birkinshaw and Duke (2013) have noted that open forums have the effects of spurring individuals use their knowledge and expertise more responsively. They further argue that creating supportive culture and risk-taking opportunity are the main ends of open forums. In Wasko and Faraj (2005), and Davenport and Prusak (1998) view, risk aversion is the main cause of poor KACE. Therefore, the open forums often has a positive effect of KACE.

Competition among individuals in addressing daily challenges promotes the application of knowledge through personal engagement (Gressgård et al., 2014). The definition of competition described above and operationalised in this paper is consistent with a focus on individuals stimulation state towards actions that foster knowledge application (Birkinshaw & Duke, 2013). In fact, according to Teglborg et al. (2012), competition or tournaments among individuals provide efficient solutions for problems in hand. For example, staff members can be invited to showcase their achievement to deal with a complex IT incidents and inspire their colleagues (Birkinshaw & Duke, 2013). Such means could also spur other staff members to use their personal knowledge (tacit knowledge) effectively in order to address a given challenge (Sarin & McDermott, 2003). Recent studies (e.g., Conger & Probst, 2014; Trusson et al., 2014) suggest that further research is warranted to understand why IT staff members are reluctant to apply their personal knowledge in addressing IT incidents.

All of the aforementioned factors affect KACE in organisations (Birkinshaw & Duke, 2013; Conger & Probst, 2014; Gressgård et al., 2014; Trusson et al., 2014). Each Factor is expected to foster the
application of existing knowledge (i.e. SKMS and personal knowledge) allowing individuals to use that knowledge to achieve the organisational goals. Moreover, our focus in this paper is on KACE in IT service desk function rather than the organisational objectives. Thus proposes the following assumptions:

**H1:** The use of time-out in IT SD function is positively associated with KACE.

**H2:** The use of expansive roles in IT SD is positively associated with KACE.

**H3:** The use of open forums in IT SD is positively associated with KACE.

**H4:** The use of competition among employees IT SD is positively associated with KACE.

### 1.2. Methodology (study context and data collection)

The empirical study was carried out using semi-structured interviews with IT in HESA. Those managers were chosen as a respondents as they should be able to provide data required for the purpose of the paper. The interviews were intended to confirm or refute the assumptions made based on the literature review. These interviews, conducted between October and December 2016, obtained inputs from 23 IT serviced desk managers in three groups: IT service desk team leaders (12 individuals), IT service desk director (8 individuals), and IT service desk deputy managers (3 individuals).

The interviews were digitally recorded and lasted between 30-35 minutes while researcher hand-wrote detailed notes as well. Each interview was transcribed verbatim and coded before the next interview took place. Four interviews were excluded from the analysis because of technical recording problems or incomplete and ambiguous answers. A total of 19 interviews were then available and provided a lot of valuable information needed for the study.

#### 1.2.1. Study analysis

Before the data analysis started, all interviews were listened several times in order to identify elements that specifically related to the study objective. The interviews were then coded and verbatim transcribed, reviewed by the interviewer himself to ensure that the transcriptions reflected the actual meaning of the interview as far as possible. The text was then analysed and categorized following the thematic approach. After that, manual content analysis was used to categorise data in themes that most frequently mentioned by participants and consistent with factors listed through the literature review. Finally, Leximanacer, a text analytics tool which produces thematic map, was used to further confirm the data analysis. After the analysis, the researcher listened to the interviews once again, to ensure all important insights had been included in the analysis.

#### 1.2.2. Results

Salient four themes were emerged from the data: providing extra time, temporary responsibility, level of competition and meeting for debate/discussion. These themes quotes are outlined below. Figures 2 shows theses four themes.

##### 1.2.2.1. Providing extra time to complete tasks/work

Many participants reported that they would use either personal knowledge or the organisational knowledge (SKMS) when they have been given extra time to in dealing with IT incidents. As one participant stated, “Okay, so, I would say that I had about maybe three or four incidents which were solved in that way in one week. The other day I was given a short period to rest and plan what I’m going to do next, suddenly I started thinking about a complex IT incident and I realised that I have the knowledge to address them—immediately I went and handled the situation very well. Then I found myself
in some situations where I wasn’t using that knowledge during the actual work hours (ITSD11). Another participate stated, “we are doing a disservice to KACE of staff members by not providing time for them to be creative” (ITSD5). This is consistent with the argument developed in the literature review for this paper. “Time-out” was a technical term to describe the extra time given to staff members to preform or handle any given IT incidents.

1.2.2.2. Temporary responsibility

Providing employees with temporary responsivity, as opposed to permanent responsibility, for dealing with complex or time consuming IT incidents in certain ways was perceived as something surprisingly positive to the KACE. Multiple participants (42.7%) indicated that they would be interested in that, “Well from my point of view this is really a useful means of persuading a staff member to their best. I think that’s very important. It’s like really something positive. If you allow me to take over responsibility for dealing with a complex IT incident, it gives me the feeling that I’m in control of things, even though it’s sometimes risky to be in such place. Anyhow, responsibility, even if it’s for short time, creates something positive and provides an opportunity for someone to unleash their power to solve too difficult problems. So, I think it’s really a positive thing (ITCDI6). In most case, using this concept to impact KACE, for the benefit of IM, was perceived as something positive and yet not used in the IT service desk function in HESA.

1.2.2.3. Level of competition

Level of competition relates to how someone tries to win something or be more successful than someone else in a given situation (e.g. addressing a complex IT incident). Participants expressed the importance of having competition in their own area of interest. As one participant sated, “competition among staff members in addressing unusual or sudden incidents does matter. Broadly speaking, competition helps most individuals to exploit their skills and use them in IT incident management“(ITSDI6). The concept of “rivalry” was also expressed as a motivating factor for some staff members. As one participants stated, “Rivalry among employees for dealing with IT problems will naturally leads to more efficient incidents management and undeniably it helps in focusing on area related to showing your high level of knowledge or skill, it is an opportunity for me to introduce myself to people” (ITSDI10). Another participant stated that the competition in this context is all about motivating people to apply what they know, “helps you stay motivated all the time” (ITSDI02). Thus, allowing IT staff members to have competition among each other in addressing a complex or time consuming IT incident seems to have considerable influence on the KACE because it can help members link their existing knowledge with real-life contexts.

1.2.2.4. Forums for debate/discussion

A situation or meeting in which people can talk about problems or matters especially of common interest was expressed by participants as another important aspect for KACE. It was noted that some forums already exist and some will evolve. As one participant stated, “meeting is truly important issue for us [...] and it can be a real eye-opener for individuals to show how they are well-versed in addressing unusual or difficult IT incidents, and oftentimes the discussions in a meeting revolves around how to deal with IT incidents, they are very helpful because just being part of a meeting allows you to talk and use your experience in front of the team” (ITSDO23). Open forums thus offer a practical means to stimulate staff members’ thoughts and activity in IT service desk function. Another participant stated that “if you want people to do their best in an IT incident, just get them together to reflect on their experiences and let
them solve all IT incidents before putting the meeting off" (ITSD03). It was noted that some form of open forums already exist. As noted by another participant, “we use forums frequently but mainly for discussion and sharing information and not for asking people to apply what they know to real problems” (ITSD07). Another participant provided an interesting response, “I have to say that when I have been invited to an open meeting to discuss a time consuming IT incident, I usually review my knowledge to make that I’ve got much savvy” (ITSD0141). In line with these response, open forums among IT service desk personnel would be beneficial for KACE.

Figure 2: KACE factors key concepts map

1.3. Conclusion, limitation and future studies
This paper has sought to show how certain factors affect KACE. A qualitative study provided insights into how four innovation factors impact KACE. However, the findings of this paper are not generalizable, but results can be transferable and have utility in that aspect. Based on results, it was concluded that EDI enablers in this study proactively affected KACE. These findings are consistent with some research studies related to how KACE can be affected (Conger & Probst, 2014; Nabiollahi, Alias, & Sahibuddin, 2011; Trusson et al., 2014). Findings articulate the importance of building a strong environment for individuals to use and practice the knowledge at the workplace. The main argument in this study strongly supported through specific comments made by participants. Four themes were emerged form data including: providing extra time, assigning a temporary responsibility, allowing competition among individuals and running open forums. Considering the value of these factors in improving KACE of IT service desk function, we urge IS researchers to move beyond those debates and deeply understand why individuals are sometimes reluctant to use their personal knowledge. Therefore, additional research is needed in this area or even study other settings (e.g. health or banking). We hope that this will be a launching pad for future studies in the area of knowledge management in ITSM.

References


Superimposed Signal Representation for Deep Learning

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Keywords: Deep Learning, Signal Representation, Auto Encoder.

Feature extraction is playing an essential role in many machine learning systems. The prevalent approach is manual feature extraction. This manual method is based on handcrafted engineering where it depends on the application type and the user’s experience; In addition to that, it usually consumes a long time to reach the best performance since it is based on the trial concept. The other approach is feature learning where the model should be capable to learn and extract features by itself and in an autonomous manner. This paper proposes a feature learning model that based on deep learning by implementing two cascaded auto-encoder layers as feature learning layers. The biosignal will be represented by using spectrogram, wavelet and wavelet packet simultaneously and feed to the cascaded encoder layers. The classifier layers will be followed for evaluating the overall system performance. K nearest neighbours, support vector machine, soft-max and discriminant analysis will be used for the classification task. K nearest neighbours will result in the superior testing accuracy, 87.63%, among the four classifiers. While Softmax will lead to the lowest testing accuracy, 81.25%, within the four hired classifiers. The added classifier fusion layer will improve the testing accuracy to reach 90.75%.

Furthermore, the window size will be changed for all employed classifiers to locate the optimum window size that results in the highest accuracy for our application. The optimal window size will be 200 mSec.

1. Introduction

Deep learning with Big Data is one of the newest tendencies to rising any digital domain signal. Deep learning denotes as a machine learning techniques that apply a supervised or unsupervised structure for machine learning hierarchical structure in deep classification, while the traditional learning methods applying a shallow-structured learning design. In recent years, deep learning has a concern from the research community due to its latest performance in many research areas such as speech recognition, computer vision, cooperative filtering, and it has been effectively used in industrial products taking improvement of the large size for digital data [1].

This paper is focusing to use the deep learning model that based on executing two cascaded auto-encoder layers as a feature learning layers, also using spectrogram, wavelet and wavelet packet to represent bio-signal. At the last, four types of classifiers to evaluating the overall system performance will be used.

To representing the data, we apply a Spectrum, wavelet and wavelet packet. Spectrum defined as a spectrum of frequency for the input signal in a graphical way [3]. It is the magnitude of the short-time Fourier transforms (STFT) which common use in time-frequency feature extraction techniques and also use wavelets and S-transform for the same purpose. Englehart, Hudgin [1] tested three different TFD features in the myoelectric pattern recognition system. It included short-time Fourier transform (STFT), wavelet transform (WT) and wavelet packet transform (WPT). They are different in how they partition
the time-frequency plane. The STFT has a fixed tilling while the WT has a variable tilling. In the WT, the aspect ratio of the tilling is proportional to the centre frequency. As for WPT, it has an adaptable tilling, which can be the best for most application. However, it is computationally heavy and time costly.

In addition, the spectrogram can extract each cycle of any bio-signal and had been used in heart cycle [4]. Auto-encoders represent a simple feature learning system where the input data transfer into output data with minimum distortion possible. Auto-encoder treats solving problems of back propagation without using a learning teacher wherever it was used the input data as a learning teacher [2].

This work is using four types of machine learning methodologies to classify ten finger movements from input surface electromyography signal (sEMG): Support Vector Machine (SVM), k-nearest neighbours (KNN), discriminate analysis (DA) and Softmax classifiers. SVM is first classifier. The SVM future deliberated briefly for regression, density estimation and classification [8]. In spite of SVM cannot minimize the input data or choose the important information, it has the intense capability to recognize patterns and perfect capability to generalizing and error tolerance. Different techniques are designed to minimize the complexity of SVM prediction by terms of SVM solution with least Kernel expansion [7]. SVM has a several classification training methods which help to give a healthier accuracy and make a right decision namely quadratic programming (QP) method, particle swarm optimizing method (PSO), quantum particle swarm optimizing method (QPSO), while the modification learning problematic of SVM is least square SVM (LS-SVM).

The second classifier is K-Nearest Neighbour (KNN), which is a supervised learning algorithm. KNN widely used text classifier for its effectiveness and simplexes. The concepts of its work called lazy learning where its training phase consists of only storage whole examples of the train as a classifier. Although of its advantages, KNN has a leakage at the modelling misfits or called inductive biases [10].

The third classifier is Discriminant analysis (DA) constructed across two main hypotheses: the first hypothesis that is all independent variables are normal (Gaussian) of spreading and it is motivating to utilize the continuous data instead of discrete data in the prognostic model. Whilst the second hypothesis only works for linear discriminant analysis, while the distinctive groups of observations for the covariance matrices expected to be equivalent (homoscedasticity). Quadratic discriminant analysis (QDA) model comes to be unstable while measured the discrete or qualitative variables. In comparison, The LDA model applies more widely than the QDA model because it does not show this type of conduct [11].

The fourth classifier called Softmax which had a various loss function. The Softmax is a binary Logistic Regression classifier generalization for multiple classes. It found that the Softmax and SVM are two popular selections classifiers. In contrast, the SVM gives the outputs as un-calibrated and probably hard to understand marks for each class, the Softmax classifier provides a slightly more instinctive output i.e. normalized class probabilities and also has a probabilistic interpretation. The equation of Softmax classifier has the function mapping keeping unchanged, reading these scores as the normalized log probabilities for each class and replacing the hinge loss with a cross-entropy loss [12].

These classifiers help promising accuracies. The classification rates increase possibility to pick up the best local accuracy in classification based fusion methods. for this purpose, Fusion of data can be approved by three stages of concepts linked to the flow classification processing: Feature Level Fusion, Data Level
Fusion, and Classifier Fusion. Some methods have been improved professionals for classifier fusion also denoted by decision fusion or mixture [13].

2. Implementation

This section presents the methodology that is followed in collecting and analysis of the surface electromyography signal from ten finger movements. Furthermore, the simulation results will be presented and discussed. Figure 1. Presents a diagram for our proposed model where the surface raw bio-signal was introduced to three signal representation spectrogram, wavelet and wavelet packet simultaneously. Then the represented bio-signal would be acted as an input to the first layer of autoencoder, and for the visualisation of deep learning concept, we added another layer of autoencoder where its output was considered as features. These learnt features were used in evaluating the performance of the suggested model by utilizing different classifiers such as support vector machine, k-nearest neighbours, discriminate analysis and softmax classifiers. Finally, a classifier fusion layer was employed to pick up the best local accuracy which led to a promising accuracy.

![Figure 1. Simplified diagram of proposed model.](image1.png)

This study aimed to present and analyse our planned model by introducing a surface electromyography signal that was previously collected by our colleague Khairul Anam. In this study, we were trying to pay attention to the feasibility of deep learning in the field of biosignal and enumerate the techniques that might be employed to develop the system behaviour. In addition to that, we needed to highlight the effectiveness of both different signal representations along with classifier fusion on enhancing suggested system performance and promoting the results to a higher accurate level.

2.1. Data Acquisition

The surface Electromyography signal was read by using FlexComp Infiniti™ device. Two sensors were placed on the forearm of the participant of type T9503M. The placement of two electrodes on participant’s forearm is as shown in figure 2. The surface electromyography signal was collected by using two channels only.

![Figure 2. Positioning of the electrodes.](image2.png)
The Electromyography signal was collected from nine participants. Each participant performed one finger movement for five seconds then had a rest for another five seconds. Each finger movement was repeated six times. The same sequence was repeated for the other finger movement till completing ten finger movements as shown in Figure 3. An amplification of the signal by 1000 was applied and a sampling rate of 2000 sample per second was implemented. The collected Electromyography signal was used to classify ten finger movements, as shown in Figure 3 via using our proposed model.

The recorded signal was filtered to ensure the removal of any noise that may be superimposed on biosignal. The testing and training accuracies were calculated for each subject individually then divided by the number of subjects.

Three and ten folded cross-validations were applied on our collected Electromyography signal. Accordingly, 2/3 of the collected data was assigned to training set while remaining 1/3 to be used by testing set for three folded. While 9/10 of the collected data was assigned to training set while remaining 1/3 to be used by testing set.

![Figure 3. Ten different finger movements.](image)

The implemented spectrogram was of 256 number of discrete Fourier transform points. Wavelet utilised the analysing wavelet of Haar type and wavelet packet exploited type sym10 of the wavelet. The main reason behind choosing the above-mentioned parameters was that they led to the best system performance.

2.2. Results

Figure 3 shows the Electromyography signal collected for ten-finger movements where the y-axis represents EMG signal in volts versus time in msec.

![Figure 4. Electromyography signal for ten finger movements](image)
Table 1 shows the results of implementing two cascaded autoencoder layers as the deep learning technique for three folded cross-validations. Table 2 shows the results of executing two cascaded autoencoder layers as the deep learning technique for ten folded cross-validations. The bio-signal was represented in spectrogram, wavelet and wavelet packet simultaneously.

<table>
<thead>
<tr>
<th>Training Accuracy</th>
<th>Testing Accuracy</th>
<th>Classifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>98.01%</td>
<td>86.55%</td>
<td>Support Vector Machine</td>
</tr>
<tr>
<td>97.58%</td>
<td>84.28%</td>
<td>Discriminant Analysis</td>
</tr>
<tr>
<td>98.78%</td>
<td>87.63%</td>
<td>K nearest Neighbours</td>
</tr>
<tr>
<td>97.35%</td>
<td>81.25%</td>
<td>SoftMax</td>
</tr>
</tbody>
</table>

**Table 1.** Training and testing accuracies for three folded cross validations for the suggested model.

<table>
<thead>
<tr>
<th>Training Accuracy</th>
<th>Testing Accuracy</th>
<th>Classifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>97.89%</td>
<td>86.21%</td>
<td>Support Vector Machine</td>
</tr>
<tr>
<td>97.32%</td>
<td>83.78%</td>
<td>Discriminant Analysis</td>
</tr>
<tr>
<td>98.01%</td>
<td>87.24%</td>
<td>K nearest Neighbours</td>
</tr>
<tr>
<td>96.86%</td>
<td>80.67%</td>
<td>SoftMax</td>
</tr>
</tbody>
</table>

**Table 2.** Training and testing accuracies for ten folded cross validations for the suggested model.

For the three folded cross-validations, the k nearest neighbours showed the highest testing accuracy while softmax showed the lowest capability of classifying the input bio-signal. Moreover, support vector machine showed a high testing accuracy while discriminate analysis showed a moderate one. The behaviour of the proposed model did not change significantly towards the ten folded cross validations as shown in table 2. The raw data was introduced directly into the first layer of the auto encoder, without summoning any signal representation, the testing accuracy was less than 50%. This low value ignited the idea of applying different types of signal representations.

Classifier fusion implementation increased the testing accuracy to reach 90.75% for three folded cross-validations and 90.50% for ten folded cross validations where the classifier fusion layer followed best local classifier technique.

The simulation time that was consumed to train the network was on the average 450 seconds However it spent 1.2 seconds to test the trained network.

The testing accuracies for different classifiers were calculated versus window size. The window size values vary from 50 milliseconds to 500 milliseconds. As shown in Figure 5, the highest testing accuracy was achieved at window size 200 milliseconds. Furthermore, the four classifiers behave in the same manner in regarding testing accuracy versus window size. Therefore, all our testing accuracies were calculated at window size 200 milliseconds to obtain highest testing accuracy.
3. Conclusion

Deep learning concept was implemented by employing two cascaded autoencoder layer. Where the raw bio signal was represented by spectrogram, wavelet and wavelet packet simultaneously then the represented signal was introduced to the first layer of autoencoder, and the output of this layer would be treated as features to be considered as the inputs of the second layer of autoencoder. Afterwards, the suggested model was evaluated by four classifiers (KNN, Softmax, SVM and DA). K nearest neighbour showed a superior performance than other three implemented classifiers where the testing accuracy reached 87.63%. Support vector machine was a good classifier as well in which the testing accuracy was 86.55%. Softmax resulted in the least classification capability as it was 81.25%. Classifier fusion improved the result a lot as the testing accuracy achieved 90%. The 200 milliseconds window size led to obtaining the highest testing accuracy. In conclusion, deep learning saved the time and effort that might be spent in manually extracting features so that the proposed model was able to learn features by itself without interfering of any handcrafted engineering algorithms.

References


Multi-gradient PSO algorithm for solving non-convex cost function of thermal generating unit under various power constraints in smart power grid

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Keywords: Multi-gradient particle swarm optimization, exploration and exploitation phases, economic dispatch, power constraints

Solving fuel cost function of thermal generating units under several power constraints, e.g., ramp rate limits and prohibited operating zones, in smart power grid is a complex problem. Because of these constraints, the cost function becomes non-convex. A novel algorithm called, multi-gradient PSO (MG-PSO) is proposed to solve such a complex problem. In MG-PSO algorithm, two phases, called Exploration and Exploitation are used. In Exploration phase, the m particles are called Explorers. In each run, the Explorers use different negative gradient to explore a new neighbourhood. Whereas, in Exploitation phase the m particles use one negative gradient that is less steep than that of the Exploration phase, to exploit the best neighborhood. This diversity in negative gradients satisfies the balance between global search and local search. The efficiency and robustness of the proposed algorithm is demonstrated using two practical power systems. The MG-PSO algorithm results are compared with the recent PSO variants. We have shown a superior performance of MG-PSO algorithm in terms of several performance measures, e.g., minimum cost, convergence rate and consistency.

1. Introduction

Solving the economic dispatch (ED) problem helps in making significant savings in smart power grid (SPG). The aim of ED is to minimize the total generation cost of on-line thermal generating units (TGUs), while satisfying SPG constraints. The practical formulation of ED problem involves a non-convex cost function due to the ramp rate limits (RRLs) and prohibited operating zones (POZs). These power constraints result in the cost curve of TGU with discontinuities and high order nonlinearities. Therefore, the exact formation of the cost function under SPG constraints gives correct details about the production cost and scheduling TGUs to meet the load demand.

In order to treat a non-convex problem with the cost function of TGU, a wide variety of the evolutionary computation techniques (ECTs) based on random search have been proposed over the last few decades. Some of ECTs include genetic algorithm (GA) [1], evolutionary algorithm (EA) [2], particle swarm optimization (PSO) algorithm [3], [4], ant colony search (ACS) algorithm [5], artificial immune system (AIS) [6], honey bee colony (HBC) algorithm [7], and firefly algorithm (FA) [8]. These techniques impose a few or no restrictions on the shape of a cost function. However, they are often prone to get trapped into local optima when applied to multiple prohibited zones.

To enhance the global search ability to solve ED problem under multiple power constraints, several ECTs have been developed in the last decade including PSO based algorithms. For example, orthogonal PSO (OPSO) algorithm has been proposed to solve ED problem under different power constraints [9], [10]. A fully decentralized approach (DE) uses three stages, one to achieve consensus among agents and the
second and third stages are used for solving ED problem [11]. The chaotic PSO (CPSO) method combines PSO with an adaptive inertia weight factor and chaotic local search to solve ED problem [12]. The anti-predatory PSO (APSO) applies anti-predatory behavior, which guides the swarm to escape from the predators [13]. The hybrid PSO wavelet mutation (HPSOWM) uses the wavelet-theory-based mutation to enhance PSO algorithm in exploration and searching for a better solution [14]. However, hybrid methods are often time-consuming due to the complex algorithm structure and finding an appropriate integration of hybrid algorithm is difficult. The random drift PSO (RDPSO) is inspired by a free electron model in the metal conductors placed in an external electric field [15]. The RDPSO uses a set of evolution equations to enhance the PSO global search ability. The simulated annealing PSO (SA-PSO) algorithm uses probabilistic jumping to prevent obtaining infeasible solution [16]. A mixed-integer quadratically constrained quadratic programming (MIQCQP) uses a bi-level branch and bound method to solve ED problem [17]. The modified PSO (MPSO) has been used for a nonconvex ED problem [18].

Some improved PSO variants in the literature (e.g., CPSO [12] and MPSO [18]) use another approach, called the inertia weight factor and time-varying inertia weight factors, respectively, as a controller on the velocity vector of each particle. The objective of both factors is the control on the impact of the previous velocity of the m particles on the current iteration. Therefore, different equations have been used to describe the weight factor. In this paper, we propose a novel algorithm called multi-gradient PSO (MG-PSO) to solve ED problem, considering the generation limits, RRLs, the POZs and transmission network loss (PL) in SPG environment.

The MG-PSO algorithm uses several negative gradients. The particle’s velocity is clearly affected according to best negative gradient among all used negative gradients. We have shown that MG-PSO algorithm is able to solve ED problem under TGU constraints quite effectively.

The rest of this paper is organized as follows. We present the problem formulation in Section 2. An explanation of MG-PSO algorithm is provided in Section 3. In Section 4, we present the application of MG-PSO algorithm to ED problem. Finally, the conclusion of this study is given in Section 5.

2. Problem Formulation

Here, we explain the cost function and the power constraints imposed on SPG involved in this study.

2.1 Objective Cost Function

The objective of an ED problem is to find the optimal allocation of output real power of on-line TGUs over a period of time in order to minimize the total generation cost while satisfying the equality and inequality power constraints [15]. The cost function can be stated mathematically as

$$\text{Minimize } F_{\text{cost}} = \sum_{j=1}^{N_{\text{gen}}} F(P_j)$$

(1)

where \(F(P_j)\) is the cost function of \(j\)th TGU in \$/h, \(P_j\) is the output real power of \(j\)th TGU in MW, and \(N_{\text{gen}}\) is the number of on-line TGUs. The cost function of each TGU is related to the output real power delivered into SPG and specified by a quadratic function [13] as follows:

$$F(P_j) = a_j + b_j P_j + c_j P_j^2$$

(2)

where \(a_j\), \(b_j\), and \(c_j\) are the cost coefficients of \(j\)th TGU.
2.2 Power Constraints in SPG

Different power constraints imposed on TGUs in SPG used in the literature are explained below.

1) Power Balance Constraint: The equality constraint of the power balance can be stated as the total power generation equals to the load demand \( P_D \) in MW plus the transmission network loss \( P_L \) in MW. This is expressed by

\[
\sum_{j=1}^{N_{\text{gen}}} P_j - P_D - P_L = 0
\]  

(3)

The \( P_L \) is a function of the output real power of TGU and is given by [19]

\[
P_L = \sum_{j=1}^{N_{\text{gen}}} \sum_{k=1}^{N_{\text{gen}}} P_j B_{jk} P_k
\]  

(4)

where \( B_{jk} \) are known as the loss coefficients or B-coefficients. Generation Limits: The generation limits of each TGU is given by

\[
P_{\text{j,min}} < P_j < P_{\text{j,max}} \quad j = 1, 2, \ldots, N_{\text{gen}}
\]  

(5)

This requires that the power generation of each TGU remains between its minimum \( P_{\text{j,min}} \) and its maximum \( P_{\text{j,max}} \) limits.

3) Ramp Rate Limits Constraint: The operating range of all on-line TGUs is restricted by their ramp rate limits (RRLs) due to the physical limitation of TGUs [13]. In addition, TGUs cannot change their output power immediately. A change in TGU output power from one specific interval to the next cannot exceed a specified limit, as follows:

- If power generation increases, then

\[
P_j - P_j^0 \leq UR_j
\]  

(6)

- If power generation decreases, then

\[
P_j^0 - P_j \leq DR_j
\]  

(7)

where \( P_j^0 \) is the TGU output power at the previous interval and \( P_j \) is the TGU output power at current interval. The \( UR_j \) and \( DR_j \) are the up-ramp and down-ramp limits of unit \( j \), respectively, in MW/h. By substituting (6) and (7) in (5), we obtain

\[
\max \left( P_{\text{j,min}}, \left( P_j^0 - DR_j \right) \right) \leq P_j \leq \min \left( P_{\text{j,max}}, \left( P_j^0 + UR_j \right) \right)
\]  

(8)

Let us assume that,

\[
P_{j,\text{low}} = \max \left( P_{\text{j,min}}, \left( P_j^0 - DR_j \right) \right), \quad \text{and}
\]

\[
P_{j,\text{high}} = \min \left( P_{\text{j,max}}, \left( P_j^0 + UR_j \right) \right)
\]  

(9)
where $P_{j,\text{low}}$ and $P_{j,\text{high}}$ are the new lower and higher limits of unit $j$, respectively.

4) Prohibited Operating Zone Constraint: The physical limitations due to the steam valve operation or vibration in a shaft bearing of TGU may result in the generation units operating within prohibited zones (POZs) [16]. Due to presence of POZs, discontinuities are produced in the cost curve corresponding to POZs. In this case, it is difficult to determine the shape of the cost curve under POZs through actual performance testing. Therefore, the best solution is, the TGU that contains POZs avoids these prohibited zones. By using (5) mentioned in constraint number 2, the feasible operating zones of the $j$th TGU are given by

$$P_{j,k-1}^{m} \leq P_{j,k}^{l} \leq P_{j,k}^{u} \leq P_{j,max}$$

where $P_{j,k}^{l}$ and $P_{j,k}^{u}$ are the lower and upper bound of the $k$th POZs of the $j$th unit, and $N_{pz,j}$ is the number of prohibited zones of the $j$th unit. Incorporating these power constraints in (8), (9) and (10), we get the final set of constraints as follows:

$$P_{j,\text{low}} \leq P_{j} \leq P_{j,\text{high}}$$

$$P_{j,k-1}^{m} \leq P_{j,k}^{l} \leq P_{j,k}^{u} \leq P_{j,max} \quad k = 2, 3, ..., N_{pz,j}$$

3. Multi-Gradient PSO Algorithm

Here we briefly introduce the PSO algorithm and explain the proposed MG-PSO algorithm.

3.1 The PSO Algorithm

The PSO algorithm is a global optimization technique. The population (swarm) is distributed randomly and using iterative approach to reach global optimum. The particles inside swarm refer to the possible solutions in multi-dimensional search space. The PSO algorithm depends on; firstly, each particle flying in the search area adjusts its flying trajectory according to two guides, its personal experience and its neighborhood’s best experience. Secondly, when seeking a global solution, each particle learns from its own historical experience and its neighborhood’s historical experience. In such a case, a particle while choosing the neighborhood’s best experience uses the best experience of the whole swarm as its neighbor’s best experience. This PSO algorithm is named, global PSO [3], [4], because the position of each particle is affected by the best-fit particle in the entire swarm. The following steps explain the learning strategy of the PSO algorithm mentioned and described in [4] that is the original PSO.

**Step 1:** Let us consider a swarm population with $m$ particles ($N_{\text{particle}} = m$) searching for a solution in $d$-dimensional space, where $m > 1$. The objective of the PSO algorithm is to minimize an objective function $F(P)$. 
Step 2: Each particle \( i \) \((i = 1, 2, \ldots, m)\) in the swarm has one \( d \)-dimensional velocity vector \( V_i \) and one \( d \)-dimensional position vector \( X_i \) are given by

\[
V_i = [v_{i1}, v_{i2}, \ldots, v_{id}] \\
X_i = [x_{i1}, x_{i2}, \ldots, x_{id}]
\]

Step 3: For each particle \( i \), evaluate the objective function \( F(P_j) \) using the position vector \( X_i \).

Step 4: The \( G_{i,pers} \) is a personal position vector of particle \( i \) that is obtained by evaluating the objective function \( F(P) \). The \( G_{i,pers} \) is given by

\[
G_{i,pers} = [g_{ip,1}, g_{ip,2}, \ldots, g_{ip,d}]
\]

Step 5: Determine the global best position vector, \( G_{best} \). The \( G_{best} \) is a best particle’s position vector among all personal positions vectors of whole swarm. The \( G_{best} \) is obtained by a solution that corresponds to lowest value of the \( m \) evaluated objective functions. The \( G_{best} \) is given by

\[
G_{best} = [g_{b,1}, g_{b,2}, \ldots, g_{b,d}]
\]

Step 6: Consider the total number of iterations, \( N_{iter} \). In iteration \( t \), \( t = 1, 2, \ldots, N_{iter} \), a particle’s velocity and position vectors are updated as follows:

\[
V_i(t) = V_i(t-1) + c_1 r_{i1} (G_{i,pers}(t-1) - X_i(t-1)) + c_2 r_{i2} (G_{best}(t-1) - X_i(t-1))
\]

\[
X_i(t) = X_i(t-1) + V_i(t)
\]

where \( c_1 \) and \( c_2 \) are coefficients whose values are chosen experimentally from \([0, 2.5]\). The \( r_{i1} \) and \( r_{i2} \) are two randomly generated values within the range \([0, 1]\).

Step 7: Each particle \( i \) is evaluated using the objective function \( f(x) \) and using the position vector \( X_i(t) \) (18).

Step 8: In every iteration, the \( G_{i,pers} \) and \( G_{best} \) are updated according to (19) and (20).

\[
G_{i,pers}(t) = \begin{cases} 
G_{i,pers}(t-1) & \text{if } f(X_i(t)) > f(G_{i,pers}(t-1)) \\
X_i(t) & \text{if } f(X_i(t)) \leq f(G_{i,pers}(t-1))
\end{cases}
\]

\[
G_{best}(t) = \min \{G_{i,pers}(t)\}
\]

Step 9: Finally, at the end of iteration, the optimal solution of \( F(P) \) is given by the global best position vector, \( G_{best}(t) \) (20).

3.2 The Proposed MG-PSO Algorithm

We propose an algorithm, called the multi-gradient PSO (MG-PSO). The mechanism of MG-PSO algorithm depends on the following considerations.

Consider \( m \) particles descend at a particular negative gradient at a position \( X \) after they were flying in the space in searching for food. However, the food may be few or not found in position \( X \). Therefore, they decide to change their direction to another gradient that has a steeper negative straight line within another position (e.g., position \( Y \)). The position \( Y \) may better than the position \( X \). Then, after several
times of different negative gradients, the m particles obtain a best position that corresponds a best
negative gradient among all used negative gradients. This diversity in gradients (multiple gradients)
generates steeper and less steep slopes. In such a case, the m particles have gained ability to coverage
larger search space area. Subsequently, the m particles are guaranteed find the food.

Let us consider grad, i = 1, 2, ..., N\text{grad} are negative gradients. In each grad, we introduce two variables,
the first variable called, time and denoted by (t). The t represents the iteration (t = 1, 2, ..., N\text{iter}). The
second variable called velocity decay factor (vdf). The vdf decreases progressively with increase in t. The
change in t is \Delta t and the change in vdf is \Delta vdf. The negative gradients grad, are given by

\[
grad_i = \frac{\Delta vdf(t)}{N} \quad i = 1, 2, ..., N\text{grad}
\]

where N\text{grad} is number of negative gradients. The vdf at t, is given by

\[
vdf(t) = vdf_{\text{initial}} \times (1 - \frac{t}{N\text{iter}}) + vdf_{\text{final}} \times (\frac{t}{N\text{iter}})
\]

where vdf\text{initial} and vdf\text{final} are real and positive numbers within a range [0, 1] and vdf\text{initial} > vdf\text{final}. The
following steps explain the learning strategy of MG-PSO algorithm.

Steps 1-5: Same as PSO algorithm as in Section IIIA.

Step 6:

For i = 1, 2, ..., N\text{grad}

Choose a set of vdf\text{initial} and vdf\text{final} for each gradient grad, i = 1, 2, ..., N\text{grad}.

Choose number of iterations N\text{iter}, t = 1, 2, ..., N\text{iter}.

Determine grad, i = 1, 2, ..., N\text{grad}, using (21).

For each iteration, update the particle’s velocity and position vectors as follows:

\[
V_i(t) = vdf(t) \cdot V_i(t-1) + c_1 \cdot r_1 \cdot (G_{\text{pers}}(t-1) - X_i(t-1)) + c_2 \cdot r_2 \cdot (G_{\text{best}}(t-1) - X_i(t-1))
\]

\[
X_i(t) = X_i(t-1) + V_i(t)
\]

where c_1 and c_2 are coefficients whose values are chosen by trail and error method from [0, 2.5].
However, the best values of c_1 and c_2 depend mainly on the experimental test. The r1i and r2i are two
randomly generated values with range [0, 1].

Evaluate the particle’s performance by substituting (24) in the objective function \(F(P)\).

Determine \(G_{\text{pers}}(t)\), \(G_{\text{best}}(t)\) using (19) and (20), respectively.

End For

Repeat for all negative gradients.

End For

Step 7: Select the negative gradient with the chosen vdf\text{initial} and vdf\text{final} that gives the best fitness value.
It may be seen from (17) and (23) that the basic difference between PSO and MG-PSO algorithms lies on the update procedure of $V_i$. In PSO algorithm while updating, the previous value of $V_i$ is added with the guidance obtained from its own personal experience and the global experience. Whereas, in the case of MG-PSO algorithm, the $vdf$ of the best negative gradient is multiplied with the previous value of $V_i$ while adding with the personal experience and the global experience. The multiplication of $vdf$ to the velocity diminishes the contribution of previous velocity while updating and thus improves the performance of PSO algorithm. The $vdf$ is introduced in the MG-PSO algorithm in order to better control the $m$ particles’ ability to exploration and exploitation.

4. Application of MG-PSO Algorithm to ED Problem

Here we describe the simulation results carried out on two power systems with several SPG constraints.

4.1 Test Case 1: Power system with 6 TGUs (PS-1)

1) Details of PS-1
The PS-1 consists of 6 TGUs, 26 buses and 46 transmission lines [15]. This system is a small-scale with six dimensions for its ED problem. There are 12 POZs of 6 TGUs, which yield 13 inequality constraints according to (12). The data of PS-1, TGU capacity and coefficients, RRLs and POZs of TGU and B-coefficients were listed in [15]. At steady-state operation, the maximum load demand is 1,263 MW. The computations are achieved with 100 MVA base capacity.

2) Comparison in terms of fitness values
In [15], the 10 ECTs (e.g., GA, DEA, ACSA, AIS, HBC, FA, CPSO, APSO, HPSOWM and RDPSO) that have been tested on PS-1 are listed in Table 1. In addition, the fitness values of [11], [16] and [17] are also presented in Table 1. In addition to our proposed MG-PSO algorithm to solve ED problem under SPG constraints, we tested PS-1 by PSO algorithm. Therefore, the total ECTs that have been tested by PS-1 are 15. The PS-1 is a small-scale and it is easy for MG-PSO algorithm to obtain the global optimum. Thus, we select only two negative gradients $N_{grad} = 2$ (one for Exploration phase and another for Exploitation phase) with a set of initial and final of $vdf$ that corresponds to $grad_i, i = 1, 2$. The set parameters of MG-PSO algorithm are $[grad_1 = -0.09, vdf_{initial} = 1, vdf_{final} = 0.1], [grad_2 = -0.07, vdf_{initial} = 1, vdf_{final} = 0.3]$. The other parameters were used out on the same with PSO algorithm, $c_1 = c_2 = 2, N_{particle} = 20, d = 6, N_{run} = 25$, and $N_{iter} = 3,000$ in each independent run. The results in Table 1 show that the MG-PSO algorithm provides the best result in terms of the minimum, maximum and mean costs and has lowest standard deviation ($\sigma$) when compared with PSO algorithm and other ECTs. This gives evidence that the MG-PSO algorithm is more stable and robust.

3) Convergence characteristics of MG-PSO and PSO algorithms
Figure 1 shows the convergence characteristics of MG-PSO and PSO algorithms. Fig. 1 (a) shows average of the mean cost over 25 independent runs. The MG-PSO algorithm has better convergence properties than the PSO algorithm. Fig. 1 (b) shows the distribution of minimum costs over 25 independent runs. It shows that MG-PSO algorithm is more stable than the PSO algorithm in obtaining the global solution.

4) Comparison in terms of inequality and equality constraints
Table 2 lists the solution vector, \( P_j (j = 1, 2, \ldots, 6) \) corresponding to the best solution for MG-PSO and PSO algorithms. Both MG-PSO and PSO algorithms were able to solve the 13 power inequality constraints (12), also both algorithms avoid the 12 POZs of 6 TGUs. In addition, the MG-PSO and PSO algorithm operate within RRLs of each TGU.

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Min Cost (Sh)</th>
<th>Max Cost (Sh)</th>
<th>Mean Cost (Sh)</th>
<th>( \sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>DE</td>
<td>15,449.58</td>
<td>15,449.65</td>
<td>15,449.61</td>
<td>NA</td>
</tr>
<tr>
<td>GA</td>
<td>15,445.59</td>
<td>15,491.47</td>
<td>15,465.17</td>
<td>9.73</td>
</tr>
<tr>
<td>DEA</td>
<td>15,444.94</td>
<td>15,472.06</td>
<td>15,450.13</td>
<td>6.98</td>
</tr>
<tr>
<td>ACSA</td>
<td>15,445.30</td>
<td>15,511.52</td>
<td>15,459.51</td>
<td>12.02</td>
</tr>
<tr>
<td>AIS</td>
<td>15,446.32</td>
<td>15,481.27</td>
<td>15,456.66</td>
<td>7.39</td>
</tr>
<tr>
<td>HBC</td>
<td>15,444.58</td>
<td>15,482.39</td>
<td>15,457.94</td>
<td>8.48</td>
</tr>
<tr>
<td>FA</td>
<td>15,445.94</td>
<td>15,501.39</td>
<td>15,461.30</td>
<td>9.33</td>
</tr>
<tr>
<td>CPSO</td>
<td>15,442.98</td>
<td>15,466.39</td>
<td>15,440.12</td>
<td>5.8</td>
</tr>
<tr>
<td>APSO</td>
<td>15,445.51</td>
<td>15,538.60</td>
<td>15,473.31</td>
<td>12.90</td>
</tr>
<tr>
<td>HPSOWM</td>
<td>15,442.82</td>
<td>15,502.63</td>
<td>15,455.62</td>
<td>15.88</td>
</tr>
<tr>
<td>RCPPO</td>
<td>15,442.75</td>
<td>15,455.29</td>
<td>15,440.02</td>
<td>2.28</td>
</tr>
<tr>
<td>SA-PSO</td>
<td>15,447.00</td>
<td>15,455.00</td>
<td>15,447.00</td>
<td>2.52</td>
</tr>
<tr>
<td>MIQCQP</td>
<td>15,443.07</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PSO</td>
<td>15,447.09</td>
<td>15,449.60</td>
<td>15,447.65</td>
<td>0.56</td>
</tr>
<tr>
<td>MG-PSO</td>
<td>15,442.65</td>
<td>15,442.65</td>
<td>15,442.65</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 1. Cost Performance of the fifteen ECTs for PS-1

![Figure 1](https://example.com/figure1.png)

Figure 1. Convergence characteristics of MG-PSO and PSO algorithms for PS-1.

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Optimum output power (MW)</th>
<th>Total Output Power (MW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSO</td>
<td>( P_1, P_2, P_3, P_4, P_5, P_6 )</td>
<td>1,275.3999</td>
</tr>
<tr>
<td>MG-PSO</td>
<td>( P_1, P_2, P_3, P_4, P_5, P_6 )</td>
<td>1,275.4157</td>
</tr>
</tbody>
</table>

Table 2. Optimized Power Dispatch by Each TGU Using MG-PSO and PSO Algorithms for PS-1

Table 3 shows the comparison in terms of power balance constraint among MG-PSO algorithm and several ECTs. The load demand PD is 1,263 MW. Total output power generated as shown in Table 2 and the PL is computed using (4) and then these values are substituted in (3) to determine any mismatch from zero. The results presented in Table 3 show that MG-PSO and MIQCQP [17] algorithms have
satisfied zero mismatch in solving power balance constraint. However, other ECTs listed in Table 3, DE [11] and DRPSO [15], SA-PSO [16] and PSO techniques have obtained on mismatch closer to zero.

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Total $P_1$ (MW)</th>
<th>$P_2$ (MW)</th>
<th>$P_3$ (MW)</th>
<th>Mismatch (MW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA-PSO [16]</td>
<td>1.275.7000</td>
<td>1.263</td>
<td>12.7330</td>
<td>-0.0330</td>
</tr>
<tr>
<td>MIQCP [17]</td>
<td>1.275.4400</td>
<td>1.263</td>
<td>12.4400</td>
<td>0.0000</td>
</tr>
<tr>
<td>PSO</td>
<td>1.275.3999</td>
<td>1.263</td>
<td>12.4000</td>
<td>-0.0001</td>
</tr>
<tr>
<td>MG-PSO</td>
<td>1.275.4157</td>
<td>1.263</td>
<td>12.4157</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

Table 3. Power balance constraint results of six ECTs for PS-1

4.2 Test Case 2: Power system with 15 TGUs PS-2

1) Details of PS-2

The PS-2 is a medium-scale power system with 15 TGUs whose characteristics and the data are taken from [20]. The maximum load demand of the PS-2 at steady-state operation is 2,630 MW. The dimension of this ED problem is $d = 15$. The PS-2 has a total of 11 POZs of 4 TGUs. Thus, there are 12 inequality power constraints (12) for this ED problem. Compared to PS-1, the ED problem of this system is relatively harder to be optimized.

2) Comparison in terms of fitness values

Eight ECTs as well as our proposed MG-PSO and PSO algorithms are listed in Table 4. These 10 optimization techniques are tested on PS-2. We choose 3 negative gradients $N_{grad} = 3$ (two for Exploration phase and one for Exploitation phase) with a set of initial and final of $vdf$ that corresponds to $grad_i$, $i = 1, 2, 3$. The set parameters of the MG-PSO algorithm are \([grad_1 = -0.09, vdf_{initial} = 1, vdf_{final} = 0.1], [grad_2 = -0.07, vdf_{initial} = 1, vdf_{final} = 0.3], [grad_3 = -0.05, vdf_{initial} = 1, vdf_{final} = 0.5]\), respectively. The other parameters of MG-PSO and PSO algorithms are same as in Section 4.1(2).

The results presented in Table 4 show that the MG-PSO algorithm achieves the best result in terms of the minimum, maximum and mean cost and has lowest $\sigma = 0.15$ compared with PSO algorithm and other eight ECTs. This indicates that the MG-PSO algorithm is more stable and robust. NA represents that the results are not available in corresponding reference.

3) Convergence characteristics of MG-PSO and PSO algorithms

The convergence characteristics of MG-PSO and PSO algorithms are shown in Fig. 2. Figure 2 (a) shows average of the mean cost over 25 independent runs. The MG-PSO algorithm is better than PSO algorithm in terms of convergence properties. The distribution of minimum costs over 25 independent runs shown in Fig. 2 (b) shows that MG-PSO algorithm is more stable in obtaining the optimum solution than the PSO algorithm.
Table 4. Cost Performance of The Ten ECTs for PS-2

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Min. Cost (S/h)</th>
<th>Max. Cost (S/h)</th>
<th>Mean Cost (S/h)</th>
<th>σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPSO [10]</td>
<td>32,669.00</td>
<td>32,699.00</td>
<td>32,688.00</td>
<td>7.21</td>
</tr>
<tr>
<td>ACSA [15]</td>
<td>32,863.17</td>
<td>33,256.28</td>
<td>33,120.02</td>
<td>86.16</td>
</tr>
<tr>
<td>AIS [15]</td>
<td>32,895.91</td>
<td>33,132.01</td>
<td>33,017.65</td>
<td>58.12</td>
</tr>
<tr>
<td>HBC [15]</td>
<td>32,789.23</td>
<td>33,301.49</td>
<td>33,030.86</td>
<td>69.79</td>
</tr>
<tr>
<td>FA [15]</td>
<td>32,898.01</td>
<td>33,310.72</td>
<td>33,116.90</td>
<td>96.38</td>
</tr>
<tr>
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<td>32,652.33</td>
<td>32,959.79</td>
<td>32,744.58</td>
<td>82.47</td>
</tr>
<tr>
<td>DEA [15]</td>
<td>32,718.82</td>
<td>33,213.31</td>
<td>32,966.43</td>
<td>110.32</td>
</tr>
<tr>
<td>SA-PSO [16]</td>
<td>32,708.00</td>
<td>32,789.00</td>
<td>32,732.00</td>
<td>NA</td>
</tr>
<tr>
<td>PSO</td>
<td>32,885.20</td>
<td>33,386.20</td>
<td>33,075.12</td>
<td>110.76</td>
</tr>
<tr>
<td>MG-PSO</td>
<td>32,668.09</td>
<td>32,699.32</td>
<td>32,668.98</td>
<td>0.15</td>
</tr>
</tbody>
</table>

4) Comparison in terms of inequality and equality constraints

Table 5 presents the best solution vector \( P_j (j = 1, 2, \ldots, 15) \) obtained by MG-PSO and PSO algorithms for PS-2. Both MG-PSO and PSO algorithms solve the 12 power inequality constraints in (12) by avoiding the 11 POZs of 4 TGUs. In addition, MG-PSO and PSO algorithms work within RRLs of 15 TGUs.

Table 6 shows the comparison among MG-PSO, OPSO [10], DRPSO [15], SA-PSO [16] and PSO optimization techniques in terms of power balance constraint. The load demand \( P_d \) in this case, 2,630 MW. The \( P_l \) of MG-PSO and PSO algorithms are computed by (4). The power balance results of both algorithms are obtained by (3). The power balance results shown in Table 6 appear that MG-PSO and OPSO [10] algorithms have satisfied zero mismatch within four places after decimal point. However, other ECTs, RDPSO [15], SA-PSO [16] and PSO algorithm obtained on mismatch are \(-0.0046\), \(-0.0080\), and \(0.0304\), respectively.
The proposed MG-PSO algorithm overcome all problems that made PSO algorithm inefficient to get acceptable results. In addition, the MG-PSO algorithm appeared superior in solving the economic dispatch problem under SPG constraints of small- and medium-scale power systems in SPG compared with other ECTs mentioned in the literature.

5. Conclusion

In this paper, the multi-gradient PSO algorithm has been proposed. It has been able to effectively solve 6- and 15-thermal generating unit (TGU) of economic dispatch (ED) of power considering the multiple smart power grid (SPG) constraints.

We have shown that the MG-PSO algorithm was able to solve the equality and inequality constraints including the transmission network loss of two power systems, and avoiding all prohibited operating zones and operating within ramp rate limits.

It is evident from the minimum power dispatch results that the MG-PSO algorithm has a better performance in terms of the minimum, maximum and mean costs compared to several competitive algorithms including PSO algorithm. The MG-PSO algorithm also showed better consistency and robustness.
References


A Framework for Clustering and Incremental Maintenance of Evolving Semi-Structured data

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**Keywords:** Clustering, Incremental maintenance, Semi-Structured data

Semi-Structured data has been extensively adopted in numerous web applications as a standard data exchange format. XML is one of the most popular formats on the web due to its wide usage in representing and transferring data. Most real-world applications require low latency for transferring large amount of web data between clients and the application provider. Clustering is one of the most crucial techniques for organizing the web data into groups based on their similarities. Web data clustering has been receiving a great attention in the data mining communities. However, incremental maintenance of the web data clusters is still a challenging task. In this paper, we propose a framework for clustering and incremental maintenance of XML messages. For clustering, we propose an efficient solution for grouping the messages based on the structure and content similarity. For cluster maintenance, we propose two incremental approaches for maintaining the existing clusters dynamically after receiving new messages. The experimental results on real world datasets verify the efficiency of the proposed model.

1. Introduction

Semi-Structured data has become a standard data exchange format, which provides simplicity among web applications. For instance, healthcare, transportation, and financial transactions [10]. Extensible Markup Language (XML) format has a great acceptance in various web applications due to its self-describing and flexibility features. Practically, these applications require fast response time for transmitting massive amounts of web data between clients and the application providers. The literature reveals that identifying groups of clients can potentially speed up the response time. As a consequence, clustering has become an essential technique for grouping the web data into groups based on their similarities [1, 2, 6]. Several works have proposed for clustering XML messages. However, incremental maintenance of the web data clusters is a challenging task. The generated clusters are to be maintained incrementally when new messages arrive. Clustering technique such as k-means is inapplicable in the dynamic environment due to the adoption of a fixed number of clusters (k) and recalculation for the cluster properties. To overcome the above limitations, we introduce a new framework for clustering and incremental maintenance of evolving XML web messages. The main contributions of this paper are stated as follows:

- We propose an efficient clustering model for the XML web messages. The proposed clustering model requires a less clustering time in comparison with other clustering models.
- We introduce two incremental maintenance approaches for updating the cluster properties dynamically.
• We validate the proposed framework with extensive experiments conducted on real-world datasets.

The rest of this paper is organized as follows. Section 2 presents the related work. Thereafter, we present the proposed solution in Section 3. The experimental results are presented in Section 4. Finally, the paper is concluded in Section 5.

2. Related Work

This section highlights the studies related to clustering approaches. In the literature, there is no work has been done for the incremental maintenance of the web data clusters. In general, XML clustering approaches include three basic categories: (1) content-based, (2) structure-based, and (3) content and structure-based approaches [7, 8, 9]. We focus on hybrid-based clustering approaches that consider both structure and content features of XML documents. Liu et al. (2004) [5] proposed a novel XML clustering technique called Principal Component Analysis (PCA). First, this technique extracts features from XML documents. Next, the XML documents are converted into vectors based on the importance of their features. PCA applies k-means technique to finalize the clustering process.

Al-Shammary and Khalil (2011) [3] proposed fractal clustering model for clustering XML messages by the structure and content similarity. The fractal clustering model does not require a predefined number of clusters. In the first step, the XML messages are represented as vectors by using tf.idf-weighting scheme. Then, the fractal coefficient is applied to calculate the minimum distance between a pair of XML vectors. The last step is grouping the similar vectors based on the minimum root mean square error. The proposed fractal-clustering model has outperformed the existing clustering techniques, such as k-means and PCA with k-means. However, the existing XML clustering techniques have shown some limitations. For instance, fix a number of clusters, and the lack of incremental maintenance of the XML data clusters.

3. Proposed Solution

This section clarifies the main components of the proposed framework. The framework includes two stages: (1) Clustering and (2) Incremental cluster maintenance.

3.1 Clustering

The steps of the clustering are stated as follows: (a) generating the XML vectors, (b) calculating the minimum distance between the vectors, and (c) assigning the vectors to their proper clusters. Technically, the term frequency-inverse document frequency scheme is used to generate the XML vectors. Formula 1 presents the tf.idf scheme.

\[ w_{t,id} = tf_{t,id} \times \log \left( \frac{N}{df_t} \right) \]  

(1)

The frequency of the term t in the XML message denoted by tf, while the idf measures the importance of the term in the entire set of messages denoted and N is the total number of XML messages in the dataset. After generating the structure vector and content vector, we combine these vectors to generate the final vector of a message. The generated XML vector is used to measure the similarity score. Squared
Euclidean distance is used to calculate the minimum distance between the XML vectors. Formula 2 presents the pairwise distance.

\[
dist(v_1, v_2) = \sum_{i=1}^{n} (v_1 \cdot w_i - v_2 \cdot w_i)^2
\]  
(2)

After measuring the pairwise similarity distance, we initialize the clusters for these vectors. To initialize the clusters, we use an agglomerative clustering approach to distribute the XML vectors into a set of clusters.

3.2 Incremental Cluster Maintenance

Maintenance of the clusters starts with the generating new XML vectors. Then, a decision is made to either assign the new vector to the closest cluster or initialize new cluster by calculating the distance between the new XML vector and the existing clusters. Once the new vector is assigned to its nearest cluster, the new centroid (\(c_{\text{new}}\)) will be adjusted as shown in Fig.1.

The process of adjusting the new cluster centroid is performed incrementally based on the following formula:

\[
c_{\text{new}} = \frac{|c| \times \overline{c} + \overline{v}}{|c| + 1}
\]  
(3)

Where \(|c|\) is the cardinality of a cluster \(c\) and \(c\) is the cluster centroid. We propose two approaches for maintaining the cluster properties incrementally: (1) baseline approach, and (2) improved approach. The first approach tracks the whole XML vectors in the cluster. The second approach tracks only a part of the XML vectors. We introduce a smaller radius \(r_{2c}\), such that \((r_{2c} < r_{1c})\) to divide a cluster \(c\) into two main spaces as followings:

- Safe space: it contains the vectors that reside in the second radius of a cluster \(r_{2c}\). The distance between these vectors and their centroid is less than or equal \(r_{2c}\).
- Unsafe space: it contains the vectors that reside out of \(r_{2c}\). These vectors may be unstable and should be considered in the cluster maintenance. The improved maintenance approach only tracks the XML vectors in the unstable space based on a set of maintenance operations are as follows:
  - moveOut operation, which moves the vectors from the unsafe space to the outside of the cluster.
  - moveIn operation which receives the vectors from the outside of the cluster to the unsafe space.
4. Experimental Results

We conducted the experiments on real world datasets. Specifically, we use two datasets that totally contain 8,000 XML web messages. We quantify the effects of the proposed clustering model according to the clustering time in comparison with the fractal clustering model. The clustering time is the actual processing time that covers the entire processes for grouping the XML web messages. It is a main indicator to verify the efficiency of the proposed framework. The results show that our clustering model requires less clustering time in comparison with the fractal clustering model as shown in Fig. 2. Technically, both clustering models apply tf.idf weighting scheme to generate the XML vectors. However, the fractal clustering model needs further clustering time to find the similar vectors by calculating the offset and scale values of fractal similarity.

![Figure 2. Comparison of the clustering time for the clustering models](image)

Figure 2 shows the execution time of our proposed approaches by varying the number of the inserted messages in the experimented datasets. We note that the improved approach requires a less execution time for maintaining the clusters in comparison with the baseline maintenance approach. The main reason is that the improved approach tracks only the XML vectors in the unsafe space of the clusters. While the baseline approach tracks the vectors in both spaces of the cluster.

![Figure 3. Execution time for the cluster maintenance approaches](image)

Figure 3 shows the execution time of our proposed approaches by varying the number of the inserted messages in the experimented datasets. We note that the improved approach requires a less execution time for maintaining the clusters in comparison with the baseline maintenance approach. The main reason is that the improved approach tracks only the XML vectors in the unsafe space of the clusters. While the baseline approach tracks the vectors in both spaces of the cluster.

5. Conclusion

In this paper, we introduce a new framework for clustering and incremental maintenance of XML web messages. Specifically, we generate a set of clusters based on the combination of structure and content similarity for the XML messages. Afterwards, we maintain the cluster properties incrementally based on the cluster maintenance approaches. We propose the baseline approach which tracks the vectors that reside in the safe and unsafe spaces by using two sets of maintenance processes. In order to improve the performance of the cluster maintenance, we propose the improved approach which tracks only the vectors that reside in the unsafe space by using one set of maintenance processes. For clustering, the
experiments validate the efficiency of the proposed clustering model in comparison with the fractal clustering model. For cluster maintenance, the results show that the execution time increases with the number of inserted XML messages (1k, 2k,..., 4k). However, the improved approach that depends on the lazy maintenance of cluster scheme is more efficient than baseline approach. The resultant development for the clustering and incremental maintenance of XML web messages would be able to enhance the performance of real-world web applications by reducing the required response time.

References


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Alternative Method for Measuring Blood Oxygen Saturation

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Keywords: SpO₂, Buccal Cavity, PPG

Pulse oximeters have become an important monitoring instrument in the medical field, particularly in operating theatres, to reveal the blood oxygen saturation during surgical procedures. However, conventional oximeters may suffer from poor performance on occasions due to multiple limitations that have prompted scientists to investigate alternative locations for oxygen saturation measurement. The buccal pulse oximeter described herein was designed, fabricated and investigated to confirm its capability to measure blood oxygen saturation efficiently and accurately. Two different sensor modules were either modified or fabricated to be suitable for oxygen saturation measurement in the buccal cavity. The results of this study have indicated that it is possible to obtain good quality signals from the buccal cavity. However, in highly ventilated rooms, blood oxygen saturation level measured by the buccal oximeter was always higher than the levels obtained by finger sensors. Furthermore, during a hypoxic study, the buccal oximeter exhibited a slow response to a decrease of ambient oxygen level and the obtained oxygen saturation measurements were higher than expected, which gives a possible indication of a lack of accuracy and precision for buccal pulse oximetry.

1. Introduction

For some time, pulse oximeters have been essential medical instruments in hospitals, clinics and even domestically. Easy to use, reliable and non-invasive, they can measure the pulse rate and oxygen saturation in human blood [1]. They provide medical staff with the status of the oxygen level for a patient during a surgical procedure and warn of any abnormal or low-level oxygen saturation in blood. The measurement of oxygen saturation level is a very important indication to the medical staff of the condition of critical human tissue and oxygen supply [2].

The first goal for any medically-related instrument is to obtain accurate and reliable measurements. Although conventional pulse oximeters have proved their clinical importance and performance, on some occasions they have failed to obtain accurate results, and sometimes failed to achieve measurements at all. Low peripheral perfusion (one of the major problems that could lead to erroneous or absent oximeter readings) can be caused by a number of factors such as hypothermia, medical conditions (such as Raynauld’s Disease) and certain types of medications. There are also other limitations directly related to the site of measurement, and motion artifact is a common limitation in finger oximeters. Skin pigmentation, dye and nail polish will affect the absorption of light in the nails of the fingers and could result in faulty readings [3] [4].

For these reasons, this new study was carried out to investigate whether or not it is possible to measure the blood oxygen saturation accurately on the buccal (cheek) region in order determine whether it might be a suitable alternative measurement site.
2. Method of Study

2.1. Materials
Two custom-made buccal sensors were used in this study. The reflectance-type buccal sensor module was designed in a rectangular shape of dimensions 16 x 6 mm (see Fig. 1). A silicon photodetector was used and inserted in the center of the design. A red surface-mount LED source (wavelength 660 nm) and an infra-red LED surface-mount light source (wavelength 940 nm) were soldered to opposite sides of the sensor base. A female 9-pin connector was soldered to the sensor module to enable it to be connected to any type of pulse oximeter. To prevent any direct contact between the sensor module and the skin, a disposable translucent plastic film was used to cover the sensor part inside the measurement area.

![Custom-made reflectance buccal sensor](image)

The transmittance-type buccal sensor was constructed by modifying an adult standard finger-sensor for use in the buccal region. Another standard adult finger sensor was also used in the study, for comparison purposes. The Olimex MOD-PULSE is a reference pulse oximeter system that was used to test the sensor modules and for performing this study: this instrument uses a MSP430FG439 microcontroller from Texas Instruments to control the operation of the instrument; it can be interfaced with a PC using a UEXT port. The SpO2 measurement and the pulse rate can be viewed on the system’s LCD or using a LabVIEW software-based GUI which is already installed on PC. It measures the pulse rate and the oxygen saturation in the blood, and displays the signals on the PC display.

2.2. The Study Procedure
This study is basically to observe and verify if the buccal cavity is a suitable location for measuring blood oxygen saturation when the usual measurement location is unavailable. A 33-year-old male non-smoker, healthy with no known cardiac or blood-related diseases, volunteered to test the buccal oximeter. The study included a comparison between the finger, transmittance-type buccal, and a reflectance-type sensor. First, a comparison was made between the finger and the transmitted type buccal oximeter, then between the finger and the reflectance-type oximeter, and finally between the transmittance and reflectance type buccal oximeters. The volunteer sat comfortably in a chair with minimal movement to avoid motion artifacts. A Nellcor DS-100A finger sensor was attached to the index finger of the right hand. Another Nellcor DS-100 finger sensor was modified and added to a clip to enable attachment to the corner of the mouth as a transmittance-type buccal oximeter sensor.

A third, reflectance-type custom-made buccal sensor was also attached to the corner of the mouth. In order to prevent motion artifacts, the volunteer was required to minimize swallowing and other movement when the buccal sensors were in place. The measurements were recorded when a good signal
was attained and displayed on the measurement screen. The overall duration was 30 minutes, and a measurement was saved and recorded every 60 seconds. At the end of the normal conditions test, the volunteer performed breath-holding for a period of time so that the effect of low oxygen intake on the blood oxygen saturation could be observed. The hypoxia study for the reflectance-type buccal sensor was repeated to confirm the performance of the reflectance-type buccal oximeter in the case of low blood oxygen saturation. The volunteer recovered back to his normal oxygen saturation measurement and rested for 10 minutes, then was required to hold his breath for 60 seconds.

3. Results and Discussion

3.1. Study Results
The recorded measurements collected from the three different sensors were analyzed. The initial indication of the study is that it is possible to obtain a good quality recognizable signal from the buccal region, although it might not be as accurate as the finger sensor. Fig. 2 shows a comparison between the pulse wave-graph from the finger, transmittance-type and reflectance-type oximeters.
The signal from the finger sensor was noise-free and of high amplitude. The signal level obtained from the transmittance-type buccal sensor was also of good amplitude with very little noise, but lower amplitude than that obtained from the finger oximeter. By contrast, the signal obtained from the reflectance-type buccal sensor was of very low amplitude, noisy and almost unrecognizable. Table 1 shows a comparison between the mean ($\pm$SD) of the blood oxygen saturation $\text{SpO}_2$ of the three different sensors which were used in the study.

<table>
<thead>
<tr>
<th></th>
<th>Finger-type sensor</th>
<th>Transmittance-type sensor</th>
<th>Reflectance-type sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\text{SpO}_2 \pm \text{SD}$</td>
<td>96.2 ±0.78</td>
<td>98.7±0.48</td>
<td>95.36±0.924</td>
</tr>
<tr>
<td>Heart Rate $\pm$SD</td>
<td>80.6 ± 2.63</td>
<td>81.9 ± 3.18</td>
<td>79.8 ±3.8</td>
</tr>
</tbody>
</table>

**Table 3. Results of the study**

### 3.2. Study Discussion

The study commenced with a healthy, non-smoker volunteer, sitting comfortably on a chair in a well-ventilated room. Three sensors were attached at two locations (a finger-type, a transmittance-type, and a reflectance-type oximeter sensor). As seen in Fig. 2.a and Table 1, good signal quality with high sensitivity was obtained from the finger sensor, and the mean ($\pm$SD) of the oxygen saturation was 96.2±0.78. The signal obtained from the transmittance-type buccal sensor was of lower amplitude than that of the finger sensor, with some noise apparent, but still clear and recognizable. However, the mean ($\pm$SD) of the oxygen saturation $\text{SpO}_2$ was much higher than the finger sensor (98.7 ±0.48) under the same conditions.

On the other hand, the signal obtained from the reflectance-type buccal sensor was noisy and of very low amplitude. Furthermore, it was difficult to obtain adequate signal quality from the reflectance-type buccal sensor, which had to be re-attached to different locations inside the buccal cavity in order to obtain a satisfactory signal. Despite all these problems with the reflectance-type buccal sensor, the recorded mean ($\pm$SD) of the oxygen saturation was 95.3 ±0.924, which is a little less than the recorded mean ($\pm$SD) of the finger sensor, and much less than the transmittance-type buccal sensor.
The hypoxia test was done after the normal-condition study. The volunteer was required to hold his breath for 60 seconds to observe the effect of low oxygen concentration on the signal quality and the oxygen saturation value.

Immediately prior to the hypoxia test, the SpO2 values from the finger, transmittance-type buccal, and the reflectance-type buccal sensors were 96%, 99% and 96% respectively. Then the volunteer held his breath for 60 seconds and the SpO2 value measured by the finger sensor dropped to 95 % after 20 seconds, while the SpO2 value from the transmittance-type buccal sensor was measured to be 97 % after 35 seconds from the beginning of the hypoxia test. However, the reflectance-type buccal sensor showed no decrease in its SpO2 value during the 60 seconds of the hypoxia test, but instead showed a slight increase of SpO2 value to 97%. After 10 seconds at the end of the hypoxia test, the SpO2 value recorded by the reflectance-type buccal oximeter dropped to 93% for a few seconds before increasing to 95%. The measured oxygen saturation SpO2 from the reflectance-type buccal oximeter was changing erratically; it could not measure the oxygen saturation correctly.

4. Conclusion

Pulse oximetry is a practical way to measure blood oxygen saturation, with many advantages over the alternatives. However, there are disadvantages in pulse oximeters: some limitations are critical, affecting the accuracy and reliability of the measurements. In order to overcome restrictions affecting the suitability of the measurement sites, previous studies and investigations have investigated new locations for blood oxygen saturation measurement. The clinical study of this project was to investigate the possibility of measuring blood oxygen saturation in the buccal region at normal and hypoxic conditions.

The photo-plethysmography (PPG) signal obtained from the transmittance-type buccal sensor was of good quality, with reasonable amplitude, but lower than that of the signal obtained by the finger sensor, and noisier.

On the other hand, the signal obtained from the reflectance-type buccal sensor was much lower in amplitude than the signals from the finger and the transmittance-type sensors. There was considerable noise in the signal; often the oximeter failed to distinguish the signal and failed to measure the oxygen saturation. The oxygen saturation measured by the transmittance-type sensor was much higher than the finger sensor throughout the whole study. During the hypoxia test, the oxygen saturation measurement from the transmittance-type buccal sensor was also higher than the measured value of oxygen saturation by the finger sensor during the normal conditions. The readings from the reflectance-type buccal sensor were closer to those of the finger sensor during the normal conditions but they were confusing measurements during the hypoxia test.

These results suggest that the buccal oximeter is not as accurate as a conventional oximeter. Furthermore, the reflectance-type buccal sensor failed to give a reliable measurement during the hypoxia test, which makes it an unsuitable tool to measure the blood oxygen saturation. However, it should be noted that the buccal sensors were connected to conventional oximeters, which are calibrated to take measurements from peripheral sites. With more studies on the buccal region, using the help of blood gas analysis, a new calibration equation suitable for the buccal region could be acquired, and more reliable and accurate measurements might well be obtained.
References


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Person Identification by Gait Analysis Using Photogrammetry Techniques and Foot Pressure Sensing Matt

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Keywords: Photogrammetry, Foot Pressure Sensing Matt, Forensic, Gait Analysis, Australis

A new technique is proposed in this research to help find a more secure environment and to identify the person through the gait pattern using photogrammetry and foot pressure sensing device. The approach proposed is to place 4 digital high-resolution video cameras around the entrance of a building and place the foot pressure sensing mat underneath the entrance carpet. The results pointed out that the knee to ground distances among the all three subjects performed in this study in all three portions of the gait was highly similar among the three tests for each subject themselves and was highly variable between each one of them. For subject 1 the difference in all three portions for three tests was approximately 1mm. This result is better than 6mm in average that obtained by [1]. In this research showed the new close photogrammetry system has identified the distances among the individuals with high accuracy which provided a set of data showed the difference among each person.

1. Introduction

Recently advanced surveillance technology has become an essential part of civilian protections from crimes. This is due to the increase of the crimes numbers and sophisticated criminal activities, and the need to more secured living [2]. These results in increasing the number of crime forensic investigations which promote the development of new advanced techniques that help to assist investigations and to identify the suspects. According to Jain et al., (2004) one of the powerful used tools for reliable automated person identification is biometric-based identification techniques. Due to the challenges of facial comparison because of the probability of concealment, the photogrammetry technique of gait analysis becomes crucial [3]. Recently using gait analysis in forensic science has an advantage which is a gait is hard to conceal unlike the facial features [4]. Ideally, analysing full gait pattern should be obtained when the video cameras filmed full gait cycles. A simple definition of gait cycle is that it consists two steps starts from right or (left) heel strike to the next right or (left) heel strike again. One step of gait cycle comprises heel strike, stance and toe off before it goes to the swing phase [5]. In this research, photogrammetry techniques have been used to analyse the captured images of three individuals from 4 digital cameras. One of the main function of the photogrammetry if measuring objects in three-dimensional space, using photos that taken to the object from multi-views and angles. A true scale model will apply when the cameras that are recording the objects are calibrated [1]. Camera calibration is one of the most important steps in photogrammetry research. A modified camera calibration approach that has been used to improve the accuracy of 3D measurements $1.25 \pm 0.3 \text{ mm}$ to $0.43 \pm 0.1 \text{ mm}$ [6]. Additionally, Australis version 6.06 copyright c 2004 photometrix pty limited have been used in this
research, a software package that allows the above applications [7]. In this research, a foot pressure sensing matt has been used to calculate whether the moment of the 3 parts; heel strike, mid-stance, and toe-off as shown in figure (1) by using Tekscan Pressure Measurements System Version 6.70-03 [8].

![Figure 1](attachment:image.png)

(a) heel strike (b) mid-stance (c) push off phase

**Figure 1.** One step sequence of gait cycle phases which start from (a) heel strike (b) mid-stance and (c) push off phase.

The Institute of Forensic Medicine in Copenhagen has developed an approach based on surveillance recordings to help police recognize and identify the perpetrators [3]. Their studies based on comparing the suspect’s posture and joint angles during gait with the perpetrator’s instead of combining the basic ability comprehension to figure out people with gait analysis knowledge and give statements whether a suspect is a perpetrator in some given cases. This institute has developed a set of forensic gait analysis data checklist based on side to side movements of the head and hyperextension in the knee joints. They described the characteristics of the perpetrator’s gait; then they analysed the rotations of each joint and segment movements which they have (by trial and error algorithm), thus found that as a suitable approach for forensic gait analysis [2]. According to Geradts, et al. [9]. The posture of the perpetrator during the robbery was compared to the posture of the suspect based on a covert recording supplied with the images obtained for photogrammetric use. The authors found concordances between perpetrator and suspect, such as restless stance, the anterior positioning of the head showing a neck lordosis, and inversion in the left ankle joint [9] also observed some incongruities. Thus, this paper, specifically describe and discuss a new approach to gait analysis, combined with photogrammetry and posture analysis to identify the suspect, based on the correlation of plantar pressure data with kneecap trajectory.

2. Methodology

The methodology of this research described as using 4 digital video cameras (Panasonic DMC fz 300) placed on the main entrance of a building that wanted to be secured such as banks. Also, placing foot pressure sensing matt on the floor of the same entrance (Fig. 1). The purpose of setting cameras is to record the lower limp movements of the entering people. These cameras are placed on a low high to ensure close range video capturing. Whereas, the pressure matt is used to identify the three parts of the human step (i.e. heel strike, mid stance and push off) while walking. Thus, obtaining some physical measurements of individuals by using photogrammetry techniques through Australis Software.
The above testing technique in the following procedure has been used. First of all, calibration the 4 digital video cameras of 50 frame/second have been done and then placed them at different positions of the entrance of our lab. Next step was calibrating the space covered by the cameras. Setting the foot pressure matt, which contains 4 sensors per square centimetre, on the entrance. After that, collecting and capturing data of subjects.

The first extracting frame to be analysed is the heel strike position which is identified by the foot pressure mat. The second frame is the mid-stance position and the third one the push off position.

By using the Australis Software for each frame, calculating the 3D distance from the knee to the ground. Also, calculating the 3D distance between the two knees of each person. This process is repeated three times for each subject. Choosing to work with the knees because they are visible during walking, even though the person wearing wide pants.

3. Results

By following the procedure mentioned above, 18 (3D) distances for each person as shown in table 1 & 2, have been obtained. The similarities in the data obtained from the same person from the three tests. However, the tests have shown that there are differences between subjects as shown in table 3. The results pointed out that the knee to ground distances among the all three subjects performed in this study in all three portions of the gait was highly similar among the three tests for each subject themselves and was highly various between each one of them. For individual 1 the average difference in the all three portions of the three tests was approximately 0.0014 except the midstance which was 0.0052 as shown in table 3. Additionally, the knee to knee distance between the right and left lower limbs was up to 2mm difference in the midstance phase of the gait. For subject 2, the knee to ground distances and knee to knee distances were similar among the all the all three tests, and noticeably there was no significant difference except in the heel down phase the knee to ground distance differed by approximately 5mm between test 1, 2, 3, and the knee to knee distance in the push off phase differed by 2 mm table (2). Calculating the distances according to equation 1, where p1 represent the 3d...
coordinates \((x_1,y_1,z_1)\) of the centre of one knee, and \(p2\) represent the 3d coordinates \((x_2,y_2,z_2)\) of a lowest point on heel of the shoe. Equation (1) also applied to calculate the 3D distance between two knees of the subject. In 3D coordinates are obtained by using Australis software (7) as shown in figure (3).

\[
d(p_1, p_2) = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2 + (z_2 - z_1)^2}
\]  

(1)

![Figure 3](image)

(a) Extracting 3D coordinates of located points on the subject
(b) corresponding 3D coordinates \((x,y,z)\) of the points selected in (a).

It is worth noting that between the individual 2 and 3 in the push-off phase of the gait, the results were highly similar for the knee to ground distance and that gives an impression how the accuracy is much required in this research and minimizing the difference to less than 1mm is much required to define the suspect in case of having close values, in that case the knee to ground distance cannot be judged using one set of data for push-off phase of the gait as other phases are required to make the decision about who is the person that has the same data to the control person. By using a statistical calculation to find the differences between distances using the formula 2 which represented by table 3.

\[
d = \frac{2(V_1 - V_2)}{V_1 + V_2}
\]  

(2)

Where \(d\) represent the difference between two values, \(V_1, V_2\) represents the 3d distance in millimetre for each case (heel strike, mid-stance and push-off) for all three tests shown in table 1, 2. The results are shown in table 3:
<table>
<thead>
<tr>
<th>Individual 1</th>
<th>Knee to ground distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>600.3806444</td>
<td>609.5499696</td>
<td>743.9240425</td>
</tr>
<tr>
<td>Test 2</td>
<td>601.5982742</td>
<td>610.6334509</td>
<td>744.0685539</td>
</tr>
<tr>
<td>Test 3</td>
<td>601.0354683</td>
<td>605.8429</td>
<td>743.0778668</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Individual 2</th>
<th>Knee to ground distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>623.6194919</td>
<td>524.1590034</td>
<td>636.2002286</td>
</tr>
<tr>
<td>Test 2</td>
<td>628.7144216</td>
<td>524.1136678</td>
<td>639.2365487</td>
</tr>
<tr>
<td>Test 3</td>
<td>624.1256956</td>
<td>523.4103712</td>
<td>641.2022167</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Individual 3</th>
<th>Knee to ground distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>545.6081983</td>
<td>552.3366556</td>
<td>641.7592163</td>
</tr>
<tr>
<td>Test 2</td>
<td>546.6736737</td>
<td>551.1287436</td>
<td>641.4705066</td>
</tr>
<tr>
<td>Test 3</td>
<td>5418.372676</td>
<td>554.749153</td>
<td>639.2365014</td>
</tr>
</tbody>
</table>

Table 1. 3D distances from knee to the ground for the three individuals, three tests\individual.

<table>
<thead>
<tr>
<th>Individual 1</th>
<th>Knee to knee distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>311.6458211</td>
<td>204.559802</td>
<td>240.0672351</td>
</tr>
<tr>
<td>Test 2</td>
<td>310.9628359</td>
<td>203.8250955</td>
<td>239.3285868</td>
</tr>
<tr>
<td>Test 3</td>
<td>311.1920286</td>
<td>201.4118891</td>
<td>238.7301329</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Individual 2</th>
<th>Knee to knee distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>337.4922372</td>
<td>250.0367465</td>
<td>326.1757429</td>
</tr>
<tr>
<td>Test 2</td>
<td>338.7701054</td>
<td>248.4843524</td>
<td>324.1562148</td>
</tr>
<tr>
<td>Test 3</td>
<td>333.0571121</td>
<td>251.3114144</td>
<td>325.1759843</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Individual 3</th>
<th>Knee to knee distance (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Heel strike</td>
<td>Mid-stance</td>
<td>Push-off</td>
</tr>
<tr>
<td>Test 1</td>
<td>344.6556663</td>
<td>257.5722935</td>
<td>284.6630153</td>
</tr>
<tr>
<td>Test 2</td>
<td>345.4657247</td>
<td>256.03736</td>
<td>287.0463764</td>
</tr>
<tr>
<td>Test 3</td>
<td>347.4340776</td>
<td>255.4712582</td>
<td>289.6630167</td>
</tr>
</tbody>
</table>

Table 2. 3D distances from knee to knee distances for the three individuals, three tests\individual.
### Table 3. Difference between measurements in table 1.

<table>
<thead>
<tr>
<th></th>
<th>Heel strike</th>
<th>Mid-stance</th>
<th>Push-off</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1 - Test 2</td>
<td>0.0020</td>
<td>0.0018</td>
<td>0.0002</td>
</tr>
<tr>
<td>Test 1 – Test 3</td>
<td>0.0015</td>
<td>0.0061</td>
<td>0.0011</td>
</tr>
<tr>
<td>Test 2 – Test 3</td>
<td>0.0009</td>
<td>0.0079</td>
<td>0.0013</td>
</tr>
</tbody>
</table>

Thus, indicating that this research requires a set of measurements for future tests such as measuring the ankle and knee angle, computing the pressure distribution among the foot correlated with the knee trajectory trace, and the speed of the person. Finally, this research showed a new close-range photogrammetry approach correlated with foot pressure measurements had identified the distances among parts of subjects with high accuracy which provided a set of data showed the difference mong each person.

### 4. Discussion and Summary

In this research, some point has obtained which shows the importance of the new techniques. These points showed that minimum 4 cameras are used to capture body movements. The best part to obtain data is the mid-stance since the person is in the slowest motion. In the case of finding similarities of distance from knee to ground between subjects, it depend on the knee to knee distances. There are some difficulties of getting data appeared in increasing walking speed.

### References


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An Energy Efficient TCP DoS Attacks Mitigation Method in Cloud Computing

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Keywords: Cloud computing, TCP DoS attacks, energy consumption.

Cloud computing is a model which provides an easy, cheap, and flexible technological services. However, it poses some security problems. One of the most common security problems is the TCP DoS attack. This attack threatens any cloud in terms of energy consumption and resources exhaustion. In this paper, we propose a method to mitigate the DoS attacks in a cloud by reducing excessive energy consumption via limiting the number of packets. Instead of system shutdown, the proposed method ensures the availability of service. The proposed method can better manage the incoming packets by dropping packets from the most frequent requesting sources. This method shows that it can process 98.4\% of the accepted packets during an attack. Furthermore, it is proved that dropping the most frequent requesting sources will always save more energy than not dropping when under attacks.

1. Introduction

The importance of the cloud computing has been significantly increasing within the last decade due to the ever increasing services and facilities that cloud can deliver. In addition, cloud computing services are in extremely high demand from organizations and individuals as a result of benefits the services can offer, such as better computing power, inexpensive operation costs, good performance, and high availability. On the other hand, cloud computing projects have some security issues that need to be addressed [1]. One of the most concerning issues is the Denial of Service (DoS) attacks [2]. There are many types of DoS and distributed DoS attacks. Many of them were mentioned in [3]. One of those types is the Transmission Control Protocol (TCP) DoS flood attack, which we will discuss in this paper. The TCP DoS attacks can flood the victim’s network with extremely large numbers of packets to shut the network down and prevent legitimate users from accessing the services, to exhaust the resources and cause overheating, and to consume more energy than expected. Historically, in 2013 there was an incident of overheating Microsoft’s data center which led to 16 hours’ shutdown in Microsoft’s cloud services, such as SkyDrive, Hotmail and Outlook [4]. Therefore, developing an efficient DoS mitigation method that can keep the services available 24/7 and reduce the excessive energy consumption is an important research topic. In this paper, we propose an energy efficient TCP DoS attack mitigation method that can be implemented in the cloud environment. The proposed method is designed to keep the incoming packets number under the number of packets the server can handle. Thus, this method can improve the services for cloud projects with protecting them against the packets flood, as well as reducing cost and energy consumption.

The rest of this paper is structured as follows: Section 2 briefly describes the proposed method. In Section 3, we evaluate the performance of the proposed method. In Section 4 we sum up this paper with conclusions.
2. The Proposed Method

In this section, we discuss cloud architecture as well as the mitigation method. Our cloud includes: (1) data storage, a single server point (Server); (2) several other points that represent the legitimate points (Client); (3) spurious points (Attacker); and (4) TCP packets (TCP Packets). Assume that the Attacker uses a real, not faked or spoofed IP. We also assume that TPackets is the rate of TCP Packets, which Server can handle within a time frame Time, and LPackets is the total rate of the legitimate TCP Packets, that the Client points send to the server. The following equation is a further assumption that the rate of the legitimate packets is at most half of the handling capacity of the Server to allow for the bursts of requests:

\[ LPackets = TPackets / 2 \]  

In addition, we assume that together with the legitimate packets, the Attacker is able to send spurious to make TCP Packets more than what the Server can handle. We denote the Attacker spurious packets as S_packets, such that:

\[ LPackets + S_packets > TPackets \]  

The proposed method aims to keep the sum of LPackets and S_packets smaller than TPackets to ensure the availability of a cloud project, reduce energy consumption, and decrease the cost of the cloud maintenance:

\[ LPackets + S_packets \leq TPackets \]  

In case of a normal traffic when no attack is occurred to the cloud,

\[ S_packets = 0 \]  

From equation (2), \( LPackets = TPackets / 2 \), equation (3) is satisfied for the normal traffic scenario.

On the other hand, in the case of abnormal traffic when the attack is performed on the cloud, the proposed method \( \mathcal{P} \) will drop the TCP Packets of the most frequent requesting IP addresses in order to ensure that \( LPackets + S_packets \) does not exceed \( TPackets \). Most frequent requesting IP addresses can be filtered using any packets filtering method, such as iptables [5].

For example, suppose that our server can handle 10 packets per Time, and there are six sources of {attacker | Client} initiating a connection with the Server. The number of packets that are sent by the sources are as follows: source 1 = 1 packet, source 2 = 2 packets, source 3 = 3 packets, source 4 = 4 packets, source 5 = 5 packets, and source 6 = 6 packets. Therefore, the number of packets that are sent from sources 1 to 6 are about 21 packets, while the server can handle a maximum of 10 packets within Time. In this case, \( \mathcal{P} \) will use a filter to determine which sources are the most frequent requesting IP addresses, and then \( \mathcal{P} \) will drop them. In our example source 5 and source 6 have 5+6=11 packets, and the rest of the packets are 1+2+3+4=10 packets. Intuitively, source 5 and source 6 that have relatively higher request rates are more likely the target of the TCP DoS attacker sources than the other less frequent requesting sources. Even though there is a possibility that source 5 and source 6 could be Client sources, \( \mathcal{P} \) can keep the request rate below \( TPackets \), and the service is still available. Furthermore, using \( \mathcal{P} \) will save energy during a DoS attack as shown in the performance evaluation section below.
3. Performance Evaluation

We tested the proposed method in our lab. The network contained one Server, one Attacker, and eight legitimate Clients.

We established TCP DoS attacks from the Attacker (IP$_2$) point against the Server (IP$_1$) point alongside with the legitimate TCP connection requests from Clients (IP$_3$ to P$_{10}$). According to Tables 1 and 2, the Attacker sent 47807 requests and received only 21130 responses, and the Server sent 47893 requests and made 21219 responses.

Wireshark Network Analyser 2.0.0 [6] was used to capture the packets traversing the experimental network and no missing packets were discovered, as shown in Table I. 99.7% of the traffic with 68997 packets were using in TCP protocol. In Tables 1 and 2, sources indicate the source of a packet and destinations present the destination of a packet.

<table>
<thead>
<tr>
<th></th>
<th>Sources</th>
<th>Destinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Valid Packets</td>
<td>69235</td>
</tr>
<tr>
<td>n</td>
<td>TCP Packets</td>
<td>68997</td>
</tr>
<tr>
<td></td>
<td>Missing</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 1. Overall packets statistics

<table>
<thead>
<tr>
<th>Sources</th>
<th>Destinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rate number of packets</td>
<td>Percent</td>
</tr>
<tr>
<td>(IP$_1$) Server</td>
<td>21219</td>
</tr>
<tr>
<td>(IP$_2$) Attacker</td>
<td>47807</td>
</tr>
<tr>
<td>(IP$_3$) Client</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>(IP$_4$) Client</td>
<td>&lt; 60</td>
</tr>
<tr>
<td>(IP$_5$) Client</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>(IP$_6$) Client</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>(IP$_7$) Client</td>
<td>&lt; 60</td>
</tr>
<tr>
<td>(IP$_8$) Client</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>(IP$_9$) Client</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>Total</td>
<td>69235</td>
</tr>
</tbody>
</table>

Table 2. Sources and destination statistics

Let the probability of an accepted TCP$_{Packets}$ within the time frame of Time be $\varepsilon_1$. The number of all the incoming packets during Time is denoted as $n$. The number of TCP$_{Packets}$ which are expected to be sent to the Server during Time must not exceed $T_{Packets} \cdot$ Time. Thus, every single TCP$_{Packets}$ has the following probability to be accepted by the Server:

$$\varepsilon_1 \cdot n \leq T_{Packets} \cdot Time$$

$$\therefore \varepsilon_1 \leq \frac{T_{Packets} \cdot Time}{n}$$

In case of normal traffic when no attack is performed against the cloud, all TCP$_{Packets}$ must be accepted by the Server. As a result, the Server accept the TCP$_{Packets}$ with the following probability:
As a result, this can be expressed as follows:

\[
e_1 = \begin{cases} 
\frac{1}{Packets\cdot Time}, & \text{if } n \leq \text{Packets}\cdot Time \\
\frac{T_{\text{Packets}}\cdot Time}{n}, & \text{if } n > \text{Packets}\cdot Time 
\end{cases} \tag{7}
\]

Thus, we can claim that the proposed method $\Pi$ can process 98% of the accepted packets during an attack.

As mentioned above, that the $TCP_{\text{Packets}}$ of the most frequent requesting IP addresses will be dropped in order to ensure that $L_{\text{Packets}} + S_{\text{Packets}}$ won’t exceed $T_{\text{Packets}}$. However, this means that $L_{\text{Packets}}$ which represent legitimate users packets could be dropped as well, and not only $S_{\text{Packets}}$ has the potential to be dropped. Therefore, we must justify $\Pi$ in terms of the energy consumption, and investigate whether the benefits of $\Pi$ are sufficient enough when it is compared with the loss of $L_{\text{Packets}}$. If $\Pi$ drops $L_{\text{Packets}}$ from the traffic, it costs energy of receiving ($R_{\text{energy}}$) these packets by the Server, denoted by $\Pi_{\text{cost}}$. Whereas, if $\Pi$ drops $S_{\text{Packets}}$ from the traffic, it saves energy by not processing the packets ($P_{\text{energy}}$), as well as saving the energy $S_{\text{energy}}$ by not sending packets from a Server to Client, denoted by $\Pi_{\text{benefit}}$. According to the authors of [7, 8], $S_{\text{energy}}$ is more than $R_{\text{energy}}$. Consequently, we can formulate the relation between $R_{\text{energy}}$ and $P_{\text{energy}} + S_{\text{energy}}$ as follows:

\[
P_{\text{energy}} + S_{\text{energy}} > R_{\text{energy}} \tag{8}
\]

\[
\Pi_{\text{cost}} = n\cdot(1 - e_1)\cdot\frac{L_{\text{Packets}}}{L_{\text{Packets}} + S_{\text{Packets}}} \cdot (R_{\text{energy}}) \tag{9}
\]

\[
\Pi_{\text{benefit}} = n\cdot(1 - e_1)\cdot\frac{S_{\text{Packets}}}{L_{\text{Packets}} + S_{\text{Packets}}} \cdot (P_{\text{energy}} + S_{\text{energy}}) \tag{10}
\]

\[
\Pi_{\text{profit}} = \Pi_{\text{benefit}} - \Pi_{\text{cost}} \tag{11}
\]

where $\Pi_{\text{profit}}$ is the net energy saved when $\Pi$ is in action.

In case of no attacks $n \leq T_{\text{Packets}}\cdot Time$, from (7) the probability of packets acceptance will be $e_1 = 1$, therefore the profit of $\Pi$ will be calculated as follows:

\[
\Pi_{\text{cost}} = n\cdot(1 - 1)\cdot\frac{L_{\text{Packets}}}{L_{\text{Packets}} + S_{\text{Packets}}} \cdot (R_{\text{energy}}) = 0
\]

\[
\Pi_{\text{benefit}} = n\cdot(1 - 1)\cdot\frac{S_{\text{Packets}}}{L_{\text{Packets}} + S_{\text{Packets}}} \cdot (P_{\text{energy}} + S_{\text{energy}}) = 0
\]

\[
\therefore \Pi_{\text{profit}} = \Pi_{\text{benefit}} - \Pi_{\text{cost}} = 0, \text{ from (11)}
\]
On the other hand, in case of attacks \( n > T_{\text{packets}} \cdot \text{Time} \), from (7) the probability of packets acceptance will be \( \varepsilon_1 = \frac{T_{\text{packets}} \cdot \text{Time}}{n} \), therefore the profit of \( \Pi \) will be calculated as follows:

\[
\Pi_{\text{cost}} = n \cdot \left(1 - \frac{T_{\text{packets}} \cdot \text{Time}}{n}\right) \cdot \frac{l_{\text{packets}}}{l_{\text{packets}} + S_{\text{packets}}} \cdot (R_{\text{energy}})
\]

\[
\Rightarrow \Pi_{\text{benefit}} = n \cdot \left(1 - \frac{T_{\text{packets}} \cdot \text{Time}}{n}\right) \cdot \frac{S_{\text{packets}}}{l_{\text{packets}} + S_{\text{packets}}} \cdot (P_{\text{energy}} + S_{\text{energy}})
\]

\[
\Rightarrow P_{\text{energy}} + S_{\text{energy}} > R_{\text{energy}}, \text{from (8)}
\]

\[
\Rightarrow \Pi_{\text{benefit}} = n \cdot \left(1 - \frac{T_{\text{packets}} \cdot \text{Time}}{n}\right) \cdot \frac{S_{\text{packets}}}{l_{\text{packets}} + S_{\text{packets}}} \cdot (R_{\text{energy}}) \cdot Z \quad \text{where } Z > 1
\]

\[
\Rightarrow \Pi_{\text{profit}} = \frac{n(S_{\text{energy}} + R_{\text{energy}})}{l_{\text{packets}} + S_{\text{packets}}} \cdot \left(1 - \frac{T_{\text{packets}} \cdot \text{Time}}{n}\right) \cdot (Z - 1) \cdot R_{\text{energy}}
\]

\[
\Rightarrow Z > 1 \Rightarrow \Pi_{\text{profit}} > 0 \quad \blacksquare
\]

As a result, dropping \( TCP_{\text{packets}} \) of the most frequent requesting IP addresses when under attacks will always save more energy than not dropping.

To recap, this paper handles the security problem in cloud computing and proposes a method to mitigate TCP DoS attacks by reducing excessive energy consumption via limiting the number of packets. Instead of system shutdown, the proposed method ensures the availability of service.

4. Conclusion

To sum up, in this paper we presented an energy efficient TCP based DoS attacks’ mitigation method to enhance the security of the cloud on top of saving energy. The method maintains the availability of the service by controlling the number of the server processed TCP packets to stay below the number of packets which the server can handle. This method can reduce the energy consumption in case of a DoS attack thereby reducing the chance of shutting down services due to such attacks.

References


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Synergistic Effects of Cationic Surfactant and Silica Nanoparticles on Hydrocarbon Production from Carbonate Reservoirs

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Keywords: Nanofluid, wettability, cationic surfactant, silica, nanoparticle, carbonate

Nanofluids play an important role as prime components in a wide variety of applications including subsurface industries. Wettability alteration of oil-wet reservoirs and interfacial tension (IFT) reduction of oil/water system are key mechanisms in enhanced oil recovery. Typically, surfactant injection has used for IFT reduction and wettability restoration. Recently, however, nanoparticles have received a great deal of attention in chemical EOR. Thus, this study systematically investigates the synergistic effects of Hexadecyltrimethylammonium Bromide as a cationic surfactant and silica nanoparticles on nanofluids stability and effectiveness in terms of decane-water IFT reduction and wettability alteration of oil-wet carbonate samples. Various suspensions of surfactant-nanoparticle were formulated. Zeta potential measurements were applied to test the combined effect on suspension stability. IFT measurement was performed using pendant drop methods. While shifts in surface wettability were assessed by contact angle measurements and confirmed by spontaneous imbibition tests. Results show that increased cationic surfactant concentration can dramatically destabilize silica nanofluid. Moreover, a limited synergistic effect of surfactant-nanoparticle combinations was observed at low concentrations. Most interestingly, a drastic improvement in wettability alteration efficiency for both of surfactant and nanoparticles was recorded. The findings provide new insights into nanofluids applications in enhanced oil recovery.

1. Introduction

So-called nanofluid; a dispersion of nanoparticles (NPs) in a base fluid [1], has drawn a great attention in all research fields including subsurface projects e.g. soil-decontamination, carbon geo-storage and enhanced oil recovery (EOR). Typically, EOR processes aim to recover large amounts of trapped hydrocarbons in reservoirs after primary and secondary oil recovery methods. Carbonate reservoirs, which contains more than 50% of the discovered hydrocarbon are naturally fractured oil-wet and its capillary pressure is mainly unfavourable to hydrocarbon production. However, reducing the oil/water interfacial tension (IFT) and rendering the wettability of oil-wet formations to water-wet will are keys to facilitate the spontaneous imbibition of water to the matrix blocks expelling the hydrocarbon out of the matrix pore space. Several methods including thermal (e.g. steam injection) and chemical (e.g. surfactant flooding) techniques have been suggested to reduce IFT and to render the oil-wet or intermediate-wet surfaces to water-wet. However, the efficiency of these chemicals is very complicated depending on
surfactants properties and concentration, and the physicochemical properties of the solid surface [2]. Standnes and Austad [3] conducted a series of experiments on the effect of surfactant solution on oil recovery from carbonate reservoirs. Their results revealed that high concentration cationic surfactant solution (> the critical micelles concentration, CMC) are quite effective to support water imbibition into originally oil-wet cores. However, among all surfactant-based EOR processes at different reservoirs conditions, low surfactant adsorption on reservoir formation is the critical requirement for the economically feasible project. Further, Ma et al. [4] revealed a remarkable adsorption of cationic surfactant on carbonate surfaces which reduces the feasibility of EOR processes.

Recently, nanofluids received a growing interest in subsurface application due to its ability to render the oil-wet surfaces water-wet at ambient condition [5, 6]. Moreover, combinations of ionic surfactant-NPs have shown promising enhancement in surfactant activities [2, 7] and nano-suspension stability[8]. Silica NPs, as metal oxide form (SiO$_2$), are widely used owing to their low cost of fabrication and easy surface modification. Lan et al. [9] studied the synergistic interaction of cationic surfactant (cetyltrimethyl ammonium bromide) and silica NPs. It was found that three mechanisms were responsible for the synergistic effects including changing the NP surface hydrophobicity, controlling the aggregation of NP into aggregates with appropriate size, and reducing oil-water IFT. Recently, Al-Anssari et al. [10] investigate the effect of silica NP-anionic surfactant formulation on the wettability of oil-wet calcite at high CO$_2$ pressure. It was found that the nano-surfactant combination can effectively reduce the contact angle of oil-wet calcite/CO$_2$ system.

Despite the relevance of studies on wettability alteration, adsorption behaviour, and IFT reduction, the mechanisms and interactions between cationic surfactant-augmented silica NPs and interfaces (solid-liquid or liquid-liquid) are not fully understood particularly for oil-wet carbonate reservoir. This study, thus, investigates the effectiveness of cationic surfactant-NP combination in term of IFT reduction, wettability restoration, and stability of the nano-suspension.

2. Experimental methodology

2.1. Material

Silicon dioxide (SiO$_2$ NPs) (hydrophilic, porous spherical, Sigma Aldrich) were used to prepare different nanofluids (general properties are listed elsewhere, e.g. Al-Anssari et al. [5]). Deionized (DI) water (Ultrapure from David Gray; conductivity =0.02 mS/cm) used in this study was first equilibrated with CaCO$_3$ to avoid the potential dissolution of carbonate substrates in water during surface treatments [10]. Sodium chloride (≥99.5 mol%, Scharlan) was used to prepare brine solutions (1-20 wt% NaCl, 0.17- 4.43 M). The dissolved air was removed from brine by vacuuming for 24 hours. Hexadecyltrimethylammonium Bromide [C$_{12}$TAB, Sigma-Aldrich, ≥ 98 mol%, Mol.wt= 364.45 g.mol$^{-1}$, CMC = 350 mg.L$^{-1}$ (9.6x10$^{-4}$ mol.L$^{-1}$)] was used as cationic surfactant. N-decane (>99 mol%, from Sigma-Aldrich) was used as model oil. Stearic acid (≥98.5%, Sigma Aldrich) was used as wettability modifier to alter the wettability of originally water-wet carbonate samples (Iceland spar, from Ward’s Natural Science, as representatives for limestone formations) to strongly oil-wet [11].

2.2. Fluids Formulation

Surfactant solutions were formulated using 220/50HZ magnetic stirrer. CTAB surfactant was dissolved in water (ID water or brine) with different ratios [7]. NP suspensions were prepared via sonication of NPs in water (DI-water or brine), as a base fluid, using ultrasonic homogenizer (300 VT Ultrasonic
Homogenizer/ BIOLOGICS) for 20 minutes [5]. Surfactant-NPs dispersions were formulated by mixing surfactant to the nano-suspension and the combination was magnetically stirred for 30 minutes then sonicated for 5 minutes to formulate a homogeneous surfactant-NP suspension [12]. Pre-mixing of surfactant-nanoparticle formulation magnetically helps to decrease the sonication time and associated heat elaboration which has dramatic impacts on surfactants properties.

2.3. Surface Preparations (Cleaning and Modification to Oil-wet)
Carbonate samples were carefully cleaned to remove any potential surface fragments and inorganic contaminants. The cleaning agents and procedure were explained in details in a previously published work [5, 6, 13, 14]. Moreover, to mimic the rocks wetting properties in oil reservoir, calcite samples and limestone cores were modified to oil-wet by aging in 0.01M stearic acid (0.285g of stearic acid dissolved in 100 mL of n-decane, Al-Anssari et al. [15]) for 72 hour at 80°C. The procedure of samples modification with stearic acid has been described with details in previously published works [13, 15]. Mechanistically, carbonate ions [$Ca^{2+}$] is the primary center for stearic acid adsorption (Fig.1). Consequently, the chemisorption of stearic acid on calcite surface and thus wettability modification is controlled by the dissociation reaction of calcite which leads to increase the hydroxyl ions in the surface due to the effect of pH and dissolved CO$_2$.

![Figure 1. Surface dissociation and adsorption of stearic acid on calcite surface with the potential structure of the adsorbed layer (drawn after Mihajlović et al. [11]).](image)

The mechanisms of surface dissociation and adsorption of stearic acid is based on $H^+$ ion moving to the surface carbonate ion $CO_3^{2-}$ and stearine ions is chemisorbing on the surface center of $Ca^{2+}$ion. The $-Ca^+$ centers that result from calcite surface dissociation will be available for chemisorption.

3. Results and Discussion
3.1. Interfacial tensions of decane/CTAB-NP systems
The attractive force between oppositely charged cationic surfactants and hydrophilic silica NP controls the potential synergistic effect of this combination on interfacial properties. Thus, the IFT of the
nanoformulation-decane system expected to be complicated and strongly depended on the composition of the nanoformulation (Fig.2). The pendant drop method was used to measure IFT of the system [7].

Results show that the synergistic effect of CTAB-NP on IFT reduction is limited to the case of low CTAB concentration (> 50 mg/l). Further, when CTAB > 100 mg/l, the addition of silica NP reduces the ability of CTAB to reduce IFT of the system. Mechanistically, IFT reduction depends on adsorption of surfactant on the oil/water interface. However, the attraction between oppositely charged NP surface and CTAB head group results in a continuous adsorption of CTAB on NPs surface. Subsequently, the availability of free CTAB monomer decrease in the suspension and consequently only fewer amount of surfactant will reach the decane/water interface. The enhancement of CTAB ability to reduce IFT at low CTAB concentration is related to the Brownian motions of NP with act as a carries to the low amount of adsorbed CTAB into oil/water interface. However, the increased surfactant concentration can increase the amount of adsorbed positive monomers on NPs surface leading to neutralize the surface charges and thus screening the repulsive force between NPs (see Figure 3 left). At this stage, each collision between nanoparticles will lead coalescence which for nano and then micro aggregates that trapped a significant amount of surfactant in the bulk fluid away from the interface. Thus it is essential to understand the effect of CTAB concentration on NPs surface charges.

3.2. Stability and zeta potential (ζ) measurements
The zeta potential is a key to recognize NPs surface charge nanosuspension stability. Thus, the zeta potential of CTAB-silica nanoparticle suspensions was measured at different CTAB and nanoparticle concentrations (Fig.3, left). ζ was measured using a Zetasizer Nano ZS (Malvern Instruments, UK).
Figure 3 Left, Zeta potential of CTAB-nanoparticle suspension at different concentrations of CTAB (0 – 360 mg/l) and silica NP (0.1, and 0.2 wt %) at 23 °C and constant pH = 6.25. Right, simplified scheme of CTAB adsorption on silica NP surface showing the original negative surface charge that changed gradually to neutral and positively charge as CTAB concentration increased (drown after Al-Ansari et al. [12]).

Figure 4 shows ζ of silica nanosuspension at different CTAB and NP concentrations. Results prove the adsorption of positive surfactant on the negatively charged NPs which indicated by neutralizing and the later positively charging of NPs surfactant (see Fig.3, right). The isoelectric point (IEP), the point at which ζ = 0, depends on NP concentration in the suspension. Surprisingly, ζ values show that CTAB-silica NP suspension is unstable against aggregation and potential precipitation since ζ range was below the stability limits. Mondragon et al. [16] revealed that nanofluid can only be stable when the absolute value of |± ζ| is greater than ±30 mV.

3.3. Effect of CTAB-NP formulation on surface wettability
Although Contact angle (θ) is a global wettability evaluation approach, the complex nature and heterogeneity of pores medium can limit the validity of θ measurement since it is usually conducted on a single mineral substrates. Thus, wetness status in this study was evaluated by θ and spontaneous imbibition (SI) tests to confirm the obtained results (Fig.4). CTAB and NP concentration were based on the outcomes of the previously published data [7].

From the trends in figure 4 left, it is apparent that the surface properties of the hydrophobic calcite were changed due to the exposure to the CTAB, even with the absence of any nanoparticles. Both advancing
and receding ($\theta_r$) contact angle were reduced as surfactant concentration increase due to the formation of an adsorbed layer of surfactant monomers. However, the addition of 0.1 wt% NP to CTAB formulation drastically improve CTAB efficiency to reduce $\theta$ until reaching a constant minimum value.

SI test was performed using brine (2 wt% NaCl) as an imbibing fluid which comes into contact with the internal surface area of the porous space of the rock. When water started to imbibe, the increased weight was recorded as a function of time. The equilibrium time was reached when no change in weight was recorded [13]. Consequently, the ratio of the weight of imbibed water at any time (WT) to the weight of imbibed water at the equilibrium time (WE), and this dimension less weight was plotted as a function of time for each core sample (Fig.4 right). SI results confirmed the pre-elementary data based on $\theta$ measurements and showed that although the treatment of limestone cores with CTAB as cationic surfactant solution endorsed achieving a drastic restoration of core wettability, its performance as wettability alteration agent could be enhanced by combination with silica nanoparticles.

4. Conclusion

This study was implemented to examine efficiency of cationic surfactant-silica nanoparticles combination in terms of interfacial tension reduction, wettability alteration, and suspension stability. Following conclusions are drown from this work:

- A limited synergistic effect between cationic surfactant and silica nanoparticles on the interfacial tension of decane/water system.
- A drastic enhancement in wettability alteration of initially oil-wet carbonate substrates after treatment with surfactant-nanoparticle suspension.
- High cationic surfactant can dramatically destabilize silica nano-suspension.
- Combination of silica nanoparticles and cationic surfactant is a successful wettability alteration agent if correctly formulated.

References


Photocatalytic, Oxygen-Generating PEDOT/Nano-Ni Composite film with Sustained High Activity

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Keywords: Water splitting, Oxygen evolution reaction (OER), PEDOT, nano-Ni

Composite PEDOT films, prepared using vapour phase polymerisation incorporating with nanoparticulate Ni (‘nano-Ni’) on FTO glass have been studied as photoelectrocatalysts of water oxidation in 0.2 M Na2SO4 electrolyte at different pH. The PEDOT/nano-Ni Films proved to be active at high pH generating current densities at 0.80 V (vs Ag/AgCl) of 0.655 mA/cm² (including a photocurrent of 0.4 mA/cm²) under light illumination of 0.25 Sun. A variety of techniques were used to study and characterized the films, including: cyclic voltammetry, chronoamperograms, gas chromatography and scanning electron microscope.

1. Introduction

Renewable energy such as sunlight, wind, rain, tides, waves, and geothermal heat, are promising as alternative energy resources and could meet energy demands, combat climate change and fossil fuel depletion [1]. Sunlight-driven water splitting is a type of renewable energy that produces H2 and O2 (H2O → H2 + 1/2 O2). Its occurrence in photoelectrochemical cells (PEC) has been widely investigated since Fujishima and Honda’s discovery in 1972 of production of H2 from TiO2 [2, 3]. However, water splitting is an uphill chemical reaction that requires a standard electrochemical potential of 1.23 V vs NHE. In practice, substantially higher potentials are required due to the overpotentials for the hydrogen and oxygen evolution reactions [4]. The efficiency of water splitting systems depend on the surface area, shape and size of catalysts materials that are used [5], the electrolyte types and pH, [6] and the external biases that are applied [7]. PECs have some favourable practical outcomes including; low cost, environmental friendly, stability, wide visible absorption, and band position [8, 9].

Conductive polymers (CP) have unique properties that make them suitable for solar water-splitting applications. These include: conductivity, permeability to water, low-cost, environmental non-toxicity, electrochemical stability, useful light absorption, ready combination with other materials, excellent electron transfer properties, and uncomplicated preparative methods, amongst others. This diversity and utility impart CPs with great promise in the catalytic generation of hydrogen and/or oxygen from water under illumination by sunlight. The conversion efficiency of such “polymer” solar cells is, however, still low when compared with inorganic semiconductors [10]. Poly(3,4-ethylenedioxythiophene (PEDOT) is considered to be the best available CP in terms of conductivity, processability, transparency to visible light, stability, and fast electrochemical switching [11,12]. A significant study wherein PEDOT was used
as a “stand-alone”, light–assisted water oxidation catalyst has been carried out by Chen and co-workers [13] when they deposited PEDOT on ITO-PET sheets by vapour phase polymerization (VPP) with, and, without, the incorporation of the anionic sulfonated Mn-porphyrin (1). PEDOT without 1 exhibited a blue-white colour, while PEDOT-1 displayed a green colour having increased absorption peaks and higher photocurrent associated with O2 formation from water. One possible way to enhance PEC performance is to utilize two or more chemical materials with different spectral responses to achieve a higher overall utilization of solar energy. To the best of our knowledge no study has investigated this option as yet for water splitting PEC interfaces between Ni as nanoparticles (NP) and PEDOT. Herein, we investigate a PEC comprising PEDOT that incorporated different ratios of nano-Ni. The results confirmed that an increase of nano-Ni ratio or higher pH enhanced PEC performance. Gas Chromatography confirmed evolution of O2 gas, while EDX spectrum of scanning electron microscope image exhibited a uniform mapping distribution.

2. Experimental

2.1 Materials and method

The following materials were used: Fluorine-doped tin oxide (FTO) slides, glass microscope slides; iron(III) p-toluenesulfonate (Fe(III)-pTS); 3,4-ethylenedioxythiophene (EDOT) FTO and glass substrates were cleaned using a PLAMAFLO PDC-FMG Plasma cleaner and a DIG UV PSD PR SERIES digital UV Ozone cleaner. Sonication was carried out using a B2500R-MTH sonicator. Spin-coating onto the substrates was carried out using a WS-4008-6NPP/LITE spin-coater. Scanning Electron Microscopy (SEM) images were taken using a JEOL 7500 FESEM. pH measurements were done with an Oakton pH/conductivity meter.

2.2 Preparation of PEDOT, PEDOT/nano-Ni, on FTO-coated glass slides

Uncoated glass and FTO-coated glass slides were immersed in acetone within a TLC chamber. The baths with the immersed slides were sonicated for 90 min, the slides were washed with water and dried by blowing air over them. The FTO and uncoated glass slides were then labelled. All slides were thereafter treated in a digital ozone-UV cleaner for 20 min to remove organic contaminants. The slides were then cleaned in a plasma cleaner in order to functionalize groups on the slide’s surface with which to fix the coated chemical solutions during spin-coating. The FTO and glass substrates were heated to dryness on an IKA® RCT basic hotplate at 60 °C.

To prepare PEDOT, PEDOT/nano-Ni, 100 mg of Fe(III)-PTS was dissolved in 1.2 mL absolute ethanol. Then the required amount of Ni-nano (as applicable) was added gradually with magnetic stirring continuing for 2.5 h. The resulting dispersion (100 µL) was dropcasted onto the slide surface using a micropipette. The slides were then spun at 2000 revolutions per min (rpm) for 180 s. After spin-coating, the sample was quickly transferred to a hotplate, where it was dried at 60 oC for 15 min. Vapour phase polymerisation was carried out in a separate conical flask (500 mL capacity), equipped with a rubber stopper containing a crocodile clip suspended above the bottom of the flask. EDOT (0.450 mL) was placed in the flask and the dried, spin-coated FTO or uncoated glass substrates were held above the EDOT solution by the crocodile clip, with the stopper in place. The stoppered conical flask was then placed in an oven at 60o C for 60 min, during which time the EDOT vapour polymerised into PEDOT polymer on the slide surface. After polymerization was complete, the sample was removed, washed thoroughly with ethanol, and then left to dry overnight. The resulting dried FTO-coated samples were converted to usable electrodes by attaching a copper wire to the FTO surface with conductive silver paint.
and epoxy resin. When the silver paste was fully solidified, epoxy glue was used to cover the contact area of the wire as well as any exposed clean FTO glass surface.

2.3 Studies of PEDOT and PEDOT/Nano-Ni, on FTO as OER Photocatalysts.

PEDOT and PEDOT/nano-Ni were employed as working electrodes within a fully-enclosed quartz cell (5 x 5 x 5 cm) placed inside a closed cabinet that comprised a Faraday cage. A Pt mesh (1 x 2 cm) was used as the counter electrode. A BASi Ag/AgCl aqueous salt bridge (KCl, 3 M) served as reference electrode. The electrolyte employed was a 0.2 M Na2SO4 aqueous solution. The electrolyte was bubbled with N2 gas for 30 min before each experiment and maintained under an N2 atmosphere during the experiments. Linear sweep voltammetry (LSV), cyclic voltammetry (CV) and chronoamperograms (CA) were performed using an EDAQ466 potentiostat. Where applicable, the sample was illuminated with a Solux daylight MR16 halogen light bulb (12 V, 50 W, 240; ca. 0.25 sun intensity) with bandpass filter (315-710 nm).

2.4 Gas Analysis Studies

Gas analysis was performed via a custom-built apparatus. The apparatus comprised of a fully-enclosed electrochemical cell containing two sealed, half-cells whose electrolytes were separated only by a Nafion 117 proton exchange membrane (5 x 4 cm). The one half-cell contained the working electrode sample and a Ag/AgCl reference electrode. The other half-cell contained the Pt mesh counter electrode. One wall of the former half-cell was a quartz sheet. Illumination from the above light source was passed through the quartz sheet onto the working electrode. The incident light was filtered with the above bandpass filter. The electrodes were connected to a CHI potentiostat. The gas outlets for the working and counter electrode half cells were connected with gas-tight polymer and stainless steel tubing to sample loops connected to a dedicated Shimadzu GC-8A gas chromatograph. 0.2 M Na2SO4 adjusted to pH 12 by adding KOH.

3. Results and Discussion

3.1 Studies of PEDOT and PEDOT/nano-Ni on FTO

In the first stage of this study, we examined the cyclic voltammogram (CV) of thin-films of vapour-phase polymerized PEDOT at different pHs on FTO glass, as water oxidation electro- and/or photo-catalyst in 0.2 M Na2SO4 aqueous solution with and without illumination by light from a Solux daylight MR16 halogen light bulb (ca. 0.25 sun intensity). As can be seen in Figure 1(a), firstly, the current density and the onset potential of the PEDOT film increased with an increase of the pH. For example, the current density of the PEDOT increased five times when the pH increased from 7 to 12. Secondly, at each pH, when these films exposed to the light, the current density and the onset potential were enhanced. For example, at pH 12 when the PEDOT film was exposed to light, the current increased from 73 to 98 µA/cm2. In the second stage of this study, we examined PEDOT/nano-Ni film with a minimum amount of nano-Ni (5 mg of nano-Ni in polymerization solution)). As can be seen in Figure 1(b), these films exhibited further increases in the current density but less in the onset potential that can be seen in PEDOT film at same pH.

The next step of this research was to examine the chronoamperometric effect and the reproducibility of the photocurrents of PEC performance when the ratio of nano-Ni concentration further increased in polymerisation solution at pH 12. As can be seen in Figure 1(c), the current density increased with higher nano-Ni concentration in the polymerisation solution. The light illumination also significantly enhanced the current. For example, when 125 mg was added to polymerization solution, the resulting...
PEDOT/nano Ni film displayed an increased current density, from 0.25 to 0.65 mA/cm². While Figure 1(d) showed long-time of PEDOT/nano-Ni film operation under light for over 4 h.

![Figure 1](image1.png)

**Figure 1.** (a) CV of PEDOT under varying pH conditions. (b) CV of PEDOT/nano-Ni films with minimum concentration of nano-Ni (5 mg) with similar condition that applied in Fig.1(a). (c) Chronoamperograms of PEDOT/nano-Ni (with different nano-Ni concentration in polymerization solution) over 1 h of operation with and without light illumination of 0.25 sun (*='light on’, #='light off’). (d) Chronoamperogram over 6 h of operation under light for optimised PEDOT/nano-Ni film (125 mg of nano-Ni in polymerization solution).

### 3.2 Gas chromatograph analysis

The GC trace analysis confirmed that oxygen gas was the main gas evolved in the dark with a ratio (%) 65: 35 (O₂: H₂) and in light to be 83:17 (O₂: H₂), see Figure 2.

![Figure 2](image2.png)

**Figure 2.** GC traces gas for the optimized performance of PEDOT/nano-Ni confirm O₂ gas was the main gas that evolved.
3.3 Characterization of the PEDOT/Nano-Ni/rGO Electrode.

PEDOT/nano-Ni film morphology was investigated via scanning electron microscopy (SEM). The film was coated with 7 nm of Pt to increase the signal. Figure 3 shows an EDX-mapping peaks that confirms the distributions of Ni C, O, and S elements. It could be suggested that PEDOT/nano-Ni film when exposed to light in both CV and Chronoamperograms study, the π electrons in PEDOT were excited leaving holes at the nano-Ni surface. These holes stimulate water splitting enhances PEC performance. The GC trace confirms the oxygen evolution reaction peaks (see Figure 2).

![EDX spectrum of PEDOT/nano-Ni](image)

**Figure 3.** EDX spectrum of PEDOT/nano-Ni confirms the composition, the inset picture shows the uniform mapping distribution.

4. Conclusions

Cyclic voltammetry, photocurrent and conductivity measurements confirmed that nano-Ni particles incorporated into the conductive polymer PEDOT yielded a photocatalytic effect in which light illumination produced enhanced PEC performance. The quantity of nano-Ni present in the coating did not have a significant effect on the onset potential, but it did increase the photocurrent. In this respect, higher concentrations of nano-Ni did improve the PEC performance. It was conclude also when pH increases, the PEC performance dramatically improved. Further work be required to investigate the interfaces of reduce

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Assess the relationship between dust events and climate change using meteorological and satellite data in southern Iraq

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Keywords: Dust storms activity, aerosol index values, meteorological parameters and climate change.

Iraq is one of the countries which is most impacted by dust events in southwestern Asia, due to climatic change effects in this region. This study examines the variability of dust events and analyses aerosol index distributions with meteorological parameters. It also focuses on dust phenomena over southern region of Iraq. The aerosol index values taken from TOMS and OMI satellites and dust event records from (1997 to 2012) and data for air temperature and precipitation (1980-2012) over Nasiriyah city were analyzed using a single linear regression model and time series. The results show that the total number of dust event days were highest (3011) over Nasiriya region compared with other selected regions. In addition, important spatiotemporal variations of the monthly average of aerosol index (AI) patterns were recorded, with higher values in June (2.4) and lower values in December and January (0.9). The analysis displays a strong positive correlation between AI and air temperature (R = 0.893), while the aerosol index is negatively correlated with total precipitation (R = -0.751). Furthermore, the number of suspended dust days was the highest (1640), compared with rising dust and dust storms (1213 and 158, respectively) over Nasiriya city. The outcomes show that June and July have the most recurrent dust events in contrast the lowest recurrent were in December and January.

1. Introduction

Dust events are one of the dangerous phenomena that occur in different areas around the world. The climate system and atmosphere over both the Earth’s surface and oceans are significantly affected by dust phenomenon at a global scale [1]. Dust phenomena are generated in desert and semi-desert areas, such as the Middle East, Sahara and Mongolia. Dust activities originating in the African deserts, Asian region and the Middle East place more than 200–5000 million tons of dust into the atmosphere annually [2]. Atmospheric dust particles can impact on the radiative balance between the ground and the atmosphere by absorbing and scattering radiance[3]. Hence, dust events and aerosols have important direct and indirect impacts on the local, regional and global climate system [4-6]. During the last two decades atmospheric dust aerosols have been shown to play a significant role in determining climate change around the world because of their important influence on the thermal and solar radiation transport in the atmosphere. Aerosols also have a significant impact on hydrological processes and rainfall rate, as well as on the solar radiative balance in the earth and atmosphere system [7]. Exposure to atmospheric mineral dust phenomena have had a negative impact on people’s heath in many ways [4, 9]. Atmospheric dust events are divided into three main kinds based on visibility and wind speed. Firstly, suspended dust in which dust particles are very fine occur with a low wind speed and the visibility decreases to about 10 km. Suspended dust can remain in the atmosphere for a long period. Secondly, rising dust occurs where the wind is strong and the dust limits visibility to between 1-10 km. Thirdly, dust
storms occur as a result of strong wind speed entraining abundant dust and visibility decreases to less than 1 km [11]. A dust phenomenon is a common event in several regions around the globe, such as the Middle East areas including Iraq [12]. Because desert areas cover more than 40% of Iraq [13], most of the dust events in the Middle East come from Iraq (more than 20%). Moreover, dust in the Middle East is generated from a mixture of natural, hydrologic sources and human activities [14]. The Mesopotamian region is considered to be the second largest dust aerosol origin after the Sahara desert area, and significant numbers of dust events occur in Iraq [18]. In addition, dust events over Iraq may impact on the surrounding countries as well as the Arabian Peninsula [19]. The increasing number of drought periods and dust events have both caused a significant increase dust activities over the Arabian Peninsula, including dust events in Iraq, that correlate with shamal winds in summer [20]. Dust events occur when strong winds move small dust particles from arid regions. Dust events caused by the strong Shamal winds from the northwest carry airborne dust towards the Persian Gulf [21]. These Shamal winds move above Iraq and the Arabian Gulf throughout June to September (summer) and November to March (winter) [22]. Many studies use positive aerosol index (AI) values to approximate the concentration of atmospheric dust aerosols [23, 24] and indicate the existence of atmospheric and soil dust [25, 26]. Positive AI values can be used as a qualitative indicator for airborne absorbed aerosols such as smoke and dust while negative AI records represent non-absorbing aerosols such as sea salt and sulfate in the UV spectrum range [27]. According to the Iraqi Environment Ministry, an annual average of about 122 sand-dust storms and over 283 days with significant dust activity occur above Iraq. Therefore, researchers forecast that in the next ten years Mesopotamia might be exposed to 300 dust storms annually [28]. Due to the significant effect of dust on the economy and environment, these phenomena require urgent investigation to understand the relationships between the occurrence of atmospheric dust events and related meteorological factors over the study region. The objective of this study is to analyse the spatiotemporal distribution of dust over different regions in Iraq. It also aims to examine the correlation between AI values from TOMS and OMI satellites with dust events and some selected meteorological parameters (monthly rainfall total and mean air temperature) that are related to dust event occurrences over Nasiriya city in southern Iraq.

2. Data processing and methodology

To examine the atmospheric dust events in Nasiriya city through time, data for dust events from 1997 to 2012 (with missing data from 2001 to 2003), and rainfall and mean air temperature from 1980 to 2012 were collected from the Nasiriya Meteorological Station. AI data for the period 1997-2012 have been analyzed from the NASA TOMS and OMI satellites with a spatial resolution of (1.00° × 1.25°) to cover Nasiriya city in Iraq. To identify any correlations between aerosol index values and meteorological factors over the southern region of Iraq (Nasiriya city), the aerosol index, dust event records, mean air temperature and amount of precipitation have been plotted and analyzed statistically (SPSS) using linear regression and time series. The variability of dust events over Iraq have been examined using GIS. Then, the aerosol index values and number of dust events in every three-year period in each month are calculated and plotted for the study period.

3. The results

The outcomes of the spatial distribution of dust events are shown in Figure 1 as (a) dust storms, (b) suspended dust and (c) rising dust. These figures indicate that the pattern of total annual dust events
over Iraq is not constant. Dust events are more common in the south of Iraq while they decrease towards the northern regions.

Fig. 2 compares the different dust event types over the selected meteorological stations (Kirkuk, Khanaqin, Rutba and Nasiriya) during 1997-2012. In Nasiriya, the number of days with suspended dust was the highest at about 1640 compared with Kirkuk, Rutba and Khanaqin that had 1198, 338 and 248 days, respectively. The number of days with rising dust was also highest in Nasiriya (1213) in comparison with Rutba, Kirkuk and Khanaqin (260, 109 and 88 days). On the other hand, the number of days with dust storms was much lower compared with the other types of dust events at all stations. Thus, the highest number of dust events occurred above Nasiriya with the most prominent type being suspended dust. The total number of dust events (dust storms, suspended dust and rising dust) show a significant increase with some fluctuation over the Nasiriya region in southern Iraq during the period 1997 – 2012 (Fig. 3). The maximum number of all dust events was 330 in 2009 while the minimum number was 142 in 1998 (R²=0.76). In addition, the annual mean aerosol index distribution displays a significant increase over the study site during 1997-2012 (R²=0.59). The highest AI values were 1.82 in 2012 and the lowest values were 1.2 in 1998.

Fig. 4 shows the distribution of monthly total atmospheric dust events over the Nasiriya region for 1997-2012. As can be seen from this figure the number of dust events was highest in June and July (476 and 478, respectively), but lowest in December and January (68 and 88, respectively). The monthly mean AI (Fig. 5) showed highest values in June and July about (2.4); these months have the maximum airborne dust activity above the study area. The lowest AI values were during December and January (0.9),
showing the lowest level of performance of airborne dust processes. This study found that there is a significant positive correlation between aerosol index and all dust events with correlation coefficient of 0.93.

**Figure 4.** Monthly total of dust events in Nasiriya.  
**Figure 5.** Monthly mean aerosol index in Nasiriya.

Fig. 6 shows the annual mean air temperature and annual total rainfall covering the period 1980 to 2012. The mean air temperature shows a significant rise during the study period ($R^2 = 0.60$), while the trend of rainfall has decreased ($R^2 = 0.022$). The air temperature has a significant effect on other meteorological parameters such as wind speed and direction, relative humidity, rainfall and also it impacts on the atmospheric dust events. The monthly distribution of mean air temperature and total rainfall (Fig. 7) indicates that air temperature gradually increases from March to reach a maximum in August (38.4°C), while rainfall is greatest from November to April with the highest amount in December (24.7 mm). However, there is a lack of rainfall in summer (May, June, July and August) possibly due to the increased air temperature.

**Figure 6.** Annual total precipitation and mean air temperature in Nasiriya.  
**Figure 7.** Monthly average air temperature and total rainfall in Nasiriya (1997-2012).

Regression analysis showed a positive correlation between monthly average aerosol index and air temperature with a statistically significant (p<0.001) correlation coefficient of 0.797, while a statistically significant (p<0.005) negative relationship exists between aerosol index and annual rainfall with a correlation coefficient of 0.564. Any changes in air temperature and precipitation are represented as an indicator of climate change in given area during long-period.

4. Discussion and Conclusion

In this paper, the empirical correlations between aerosol index and selected meteorological parameters (air temperature and rainfall have been determined for the south of Iraq (Nasiriya city). The analyses display a significant and strong positive correlation ($r = 0.893$) between average monthly AI and air
temperature as well as a significant and strong negative correlation ($R = -0.751$) between average monthly AI and rainfall. This study examined how aerosol index, air temperature and precipitation were related to dust events over southern Iraq during 1997-2012. The analysis of dust events showed an increase in the number of dust events over the Nasiriya region in southern Iraq during the study period. This rise involves the total number of dusty days (suspended dust, rising dust and atmospheric dust storms). In addition, the maximum number of dust events occurred in June and July while the minimum number occurred in December and January. Annual and monthly variations in aerosol index values were examined and showed high AI values during June and July. These months also show the highest dust activity over the study region. In contrast, the lowest aerosol index values are in December and January confirming the lowest activity of dust. The annual mean aerosol index values increased significantly during 1997-2012 over the Nasiriya region. Therefore, AI values can be considered as a good indicator for the presence of dust activity in the atmosphere over the Tigris and Euphrates basin in Iraq. Moreover, this temporal distribution can be associated with other meteorological variables such as rainfall and air temperature. The pattern of mean air temperature distribution showed a significant rise during the period of study while the pattern of rainfall distribution decreased. The noticeable reduction in the annual total of precipitation amount and increase in the annual mean air temperature during the study period, both may indicate climatic change at the regional scale. Furthermore, the results showed a significant correlation between aerosol index values and the number of atmospheric dust events, annual mean air temperature and precipitation. The results displayed that monthly average aerosol index correlates positively with air temperature, while negatively correlates with rainfall amounts. Thus, dust particles are lifted into the air from dry and semi-dry areas when the annual precipitation rate is very low. Spatiotemporal variability of dust events over the southern region of Iraq was analyzed and indicated that dust activity in this region and surrounding countries is more frequent in spring and summer compared with other seasons. This study is considered as a significant document of the long period seasonal climatology of mean air temperature and precipitation amounts in Iraq for spatial distribution, that can have a link for local climate influence research because of climate change in the region. These results represent an indicator of climate change impacts in the study region. Therefore, this study brings the attention of the local government to find solutions for this problem.
References


Preparation, characterisation and 3D printing of conducting poly(acrylamide) hydrogels

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\textsuperscript{2}School of Chemistry, Nahraun University, Baghdad, Iraq
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Keywords: Hydrogels, 3D printing, mechanical and electrical characterisation.

The ability to print soft hydrogels is gathering significant interest for its potential application in the fields of soft robotics and tissue engineering. Ionic poly(acrylamide) has been prepared and characterised, before and after soaking in 6M LiCl solutions, to assess its suitability for 3D printing. Rheological analysis showed that controlled UV-crosslinking, while cooling to -6°C, enabled 3D printing of the poly(acrylamide) (PAAm) without the need for any additional rheological modifiers. The 3D printed hydrogel materials exhibited conductivity values of 117 ± 13mS/cm for the PAAm with 9M LiCl and could be stretched up to 4 times their own length.

1. Introduction

Three-dimensional (3D) fabrication techniques are attracting an attention due to their rapid prototyping capability in various applications such as self-healing materials (1), tissue engineering scaffolds (2-4), developing actuators (5-7) and sensors (8-10). 3D printing techniques can provide flexible patterning approaches using a broad range of materials, length scales, and architectures (11). Colloidal-epitaxy, standard-lithographic and direct-writing techniques have been successfully used for the precise assembly of 3D periodic arrays (12-16).

Direct ink writing is a low-cost 3D printing technique that allows for controlled patterning of a wide range of raw materials from colloidal suspensions to waxes (17-19). It may employ computer or hand-controlled devices in tandem with an ink deposition system (20). Their approaches can be divided into filamentary-based approaches such as: robocasting (21-22), robotic deposition (23), micro-pen writing (17), and fused deposition (24-25); along with droplet-based approaches such as ink-jet printing (20-21) and hot-melt printing (26). In each case, the inks are solidified through either liquid evaporation (22), photo-gelation (18, 26), or a temperature (19-20) or solvent-induced phase change (23).

Hydrogels are one of the material-types that are of current interest for the development of numerous different 3D fabrication techniques (27-28). Mechanical and electrical characterisation of gels has shown many to be highly reliable and durable; which suggests potential for this class of material in applications such as soft robotics. Furthermore, hydrogels have been shown to have highly tunable physical properties, allowing them to adapt to a wide range of applications from tissue engineering (29-30) to electronic circuitry (31). Previous research demonstrated that physically entangled, photo-polymerised poly(acrylamide) could be patterned using direct-writing techniques to prepare hydrogel scaffolds for
3T3 fibroblasts (31). Related studies focused on the preparation of ionic poly(acrylamide) using either NaCl or LiCl as the ionic pathway (29,32).

In this paper, we describe the electrical and mechanical characteristics of 3D printed poly(acrylamide) hydrogel materials. The starting materials, inks, casted and printed structures were characterized using rheology, swelling studies, electrical impedance analysis and mechanical characterisation.

2. Materials and Methods

2.1. Preparation of hydrogel

Acrylamide monomer (99.9%), N,N-methylenebis(acrylamide), sodium chloride (99.9%) and lithium chloride (99.9%) and ketoglutaric acid were purchased from Sigma Aldrich, Australia. Poly(acrylamide) hydrogel-forming solutions were synthesised as follows: First; NaCl (6, 8 or 9M) or LiCl (9M) solutions were prepared with Milli-Q water at room temperature (RT, 21°C). To these solutions, acrylamide monomer powder was added to yield a 2M acrylamide concentration. Then, 470µl of 2% w/v N, N-methylenebis(acrylamide) and 0.014(w/v) of ketoglutaric acid were added as cross-linking co-polymer and photoinitiator, respectively. The gel solution was stirred and degassed in a vacuum desiccator for 20 minutes under 0.1 Bar pressure, at room temperature. Acrylamide solutions were poured into plastic moulds and cross-linked using a Dymax BlueWave 75 Rev 2.0 UV Light at 1.15 W/cm² intensity. Gels for mechanical analysis were prepared as follows, solutions were poured into a 15 cm x 15 cm box, cured and cut into “dog-bone” shapes (conforming to JIS – K625060). Gels for electrical analysis were prepared as follows, belong hydrogel samples (height = 6mm, width = 5mm) of varying length between 5 and 25mm were cast in plastic moulds with reticulated vitreous carbon foam (RVC, ERG Aerospace, 20 pores per inch) at each end. Wires were connected directly to the RVC electrodes above the height of the gel samples.

2.2. Characterisation

2.2.1 Mechanical and Electrical characterisations

Water content of the various PAAm hydrogels was evaluated using equation 1. Masses were recorded using a digital lab balance.

\[
\text{Water content } \% = \frac{\text{Mass of hydrogel} - \text{Starting mass of hydrogel}}{\text{Starting mass of hydrogel}} \times 100
\]  

The mechanical toughness has been investigated and compared using tensile and compressive analysis under multiple cycles for PAAm+6M LiCl, PAAm+9M LiCl and PAAm+2.7M NaCl before and after soaking them in salts at room temperature (21-23°C) and 45-50% relative humidity (RH). Tensile testing was carried our using a universal mechanical tester with a 50 N load cell at an extension rate of 4mm.min⁻¹. The resulting stress-strain data was utilised for determining the elongation to failure (εf), Young’s modulus (E), tensile fracture stress (σf) and work of extension (W) (33).

Rheological analysis was performed using a rheometer (Anton Paar, Physica, MCR 301 Digital Rheometer) with a Peltier temperature-controlled stage. Viscoelastic characteristics were measured as a function of temperature and time at a constant shear rate of 100/s, using a cone and plate measuring system with 49.97 mm diameter, angle 0.992°, 0.55ml sample volume and a heat controlled sample stage at temperature range between 16 to -6°C.
Impedance measurements were obtained using a custom-built setup, described in previous studies (34). Briefly, a 1V peak voltage (alternate current signal) was applied using a waveform generator (Agilent U2761A), through the circuit with a known resistor (Rk, 10 kΩ) in series with the hydrogel sample. Impedance measurements (RI) consist of a resistance due to the sample (Rs) and a contact resistance (RC) due to the interface with the electrodes and other circuit components. In order to isolate conductivity (σ) and remove the contact resistance, the impedance behaviour of samples was measured over a series of lengths (l) with a known cross-sectional area (A); the linear slope of RI (at high frequencies) as a function of l was used to calculate conductivity according to Equation 2.

\[ R_I = \frac{1}{\sigma A} + R_C. \]  \hspace{1cm} (2)

3D printing was performed using an adapted CNC milling machine (Sherline Products, 5400) which acted as an x-y positioning stage (Figure 1a). An independent linear actuator (Zaber Technologies, T-LA60A) was mounted onto the z-axis of the milling machine and used as a syringe pump. Poly(acrylamide) materials were direct-written by depressing the syringe at a constant speed of 28 µm/sec, with stage jog speed of 134-162 mm/min. and syringe-tip height of 0.39 mm. The extruded ink is cross-linked by UV photopolymerisation during the extrusion process and between each printed layer (10 minutes). Layers are dried with nitrogen gas for 2 minutes prior to addition of the following layer. Dog-bone (width 1.5 cm, thickness 0.5 cm, length 4 cm) and rectangular (width 2.5 cm, thickness 0.5 cm) patterns were printed for mechanical and electrical characterisation, respectively.

**Figure 1.** a) Schematic of the 3D printer used to fabricate the hydrogel materials. b) A typical image of a 3D fabricated hydrogel sample in between copper electrodes.

### 3. Results and discussion

#### 3.1 Mechanical and Electrical Characteristics

PAAm-salt hydrogels of various NaCl and LiCl (with and without additional soaking) were prepared as described in the experimental section. Tensile testing was carried out on hydrogel shaped into dog bones. All gels were extended until failure (Figure 2a). PAAm hydrogels containing NaCl were seen to outperform those containing LiCl with failure point occurring at higher stresses and strains.
Figure 2. Typical examples of: (a) Stress-strain curves for ionic PAAm hydrogels with NaCl and LiCl before and after soaking (AS) in specified salt solutions, and (b) the effect of changing temperature on the viscosity (at shear rate 100 s⁻¹) of AAm+6M LiCl, AAm+8M LiCl and AAm+9M LiCl inks.

Values for modulus, strength, and extension at failure are determined. For example, PAAm-NaCl hydrogels exhibited higher ductility than their LiCl counterparts (600% extension at failure, compared to 200-400%). Decreases in sample strengths from before to after soaking can be accounted for by their increased water contents.

The conductivity was measured for each PAAm gel type (Table 1). The data for samples measured prior to soaking shows that the NaCl-containing gels demonstrate an ionic conductivity which is the same as those which contain LiCl, within experimental error (116±8 mS/cm versus 113±7 mS/cm, respectively). However, after their soaking treatments, water content was increased and PAAm-NaCl samples outperformed PAAm-LiCl by at least 50 mS/cm. This is attributed to the greater mobility of Na⁺ ions through the polymer matrix due to the shielding effect of an additional electron shell than that of Li⁺ ions, which would interact more with the polymer chains (29).

<table>
<thead>
<tr>
<th>Hydrogel</th>
<th>PAAm + 2.7 M NaCl</th>
<th>PAAm + 6 M LiCl</th>
<th>PAAm + 9 M LiCl</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Before soaking</td>
<td>After Soaking</td>
<td>Before soaking</td>
</tr>
<tr>
<td>Conductivity (mS/cm)</td>
<td>116±8</td>
<td>187±19</td>
<td>99±7</td>
</tr>
<tr>
<td>Water content (%)</td>
<td>66</td>
<td>95</td>
<td>69</td>
</tr>
</tbody>
</table>

Table 1. Conductivity and water content values for PAAm+2.7M NaCl, PAAm+6M LiCl and PAAm+9M LiCl hydrogels.

3.2 Rheology and 3D printing of ionic poly(acrylamide)

In order to determine a suitable temperature for extrusion, the flow properties of the various PAAm-salt hydrogels were measured rheometrically at a constant shear rate (100 s⁻¹) while varying the temperature from -6 to 16 °C as shown in Figure 2b. Each sample exhibited increasing viscosity with decreasing temperature, as was expected. A two-fold increase in viscosity was observed when the concentration of LiCl was increased from 6M to 9M. As such, for optimum extrusion printing, salt concentrations of AAm were maximised to sufficiently thicken the ink. To establish the rate at which the extruded materials formed a hydrogel, once patterned, the dynamic modulus was monitored as a function of UV-irradiation time using a quartz bottom plate accessory on the rheometer. PAAm-LiCl was shown to cure in 5 minutes, compared to PAAm-NaCl, which took 15 minutes.
In attempts to optimise the mechanical and the electrical properties for the hydrogel, different concentrations of LiCl have been added 6, 8 and 9M to the PAAm hydrogel. The required time to initiate photo-curing for PAAm-LiCl hydrogel was reduced with increasing salt-content due to whatever the salt content is high, the water percentage would be lower, allowing less free water molecules gelation in the polymer. Therefore, PAAm with 9M LiCl displayed the lowest gelation time with 3 minutes when compared to other LiCl concentrations 6 and 8M. Moreover, the 9M LiCl added to PAAm hydrogel revealed the lowest difference in the complex viscosities between the extruded and the molded gel.

PAAm patterns were printed with a direct extrusion printing technique using optimised AAm-LiCl (9M LiCl) inks (Figure 1b). Sample containers were covered in Aluminium foil during printing process to prevent premature curing from the UV light source and maintain temperature at (~6°C) for the duration of the print job. Glass substrates were pre-treated with paraffin oil to provide a hydrophobic surface which lessened line wetting and therefore increased feature definition.

Electrical conductivity values of printed patterns ranged from 117 ± 13 mS/cm before to 146 ± 4 mS/cm after soaking in 9M LiCl. This is within experimental error of the hand-cast samples (Table 1), indicating that the printing process was not detrimental to the conductivity.

Mechanical tensile testing of dog-bone shaped hydrogels, however, revealed a decrease of Young’s modulus and tensile strength, compared to their casted counterparts (Table 2).

<table>
<thead>
<tr>
<th>Hydrogel</th>
<th>Young’s modulus (kPa)</th>
<th>Tensile strength (kPa)</th>
<th>Extension to failure (%)</th>
<th>Work of extension (kJ/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casted PAAm+9M LiCl</td>
<td>13±2</td>
<td>22±1</td>
<td>411±18</td>
<td>50±4</td>
</tr>
<tr>
<td>3D printed PAAm+9M LiCl</td>
<td>7±0.5</td>
<td>13±3</td>
<td>435±5</td>
<td>32±4</td>
</tr>
</tbody>
</table>

**Table 2.** Mechanical properties (from tensile testing) of 3D printed and casted PAAm+9M LiCl.

4. Conclusions:

In this paper, we report the preparation, optimisation and characterisation of 3D printed ionic poly(acrylamide) hydrogels. We successfully demonstrated that the poly(acrylamide) hydrogels could be processed using 3D printing. The resulting printed hydrogel materials exhibited conductivity values of up to 117 ± 13 mS/cm (PAAm with 9M LiCl). Soaking the printed structures in 9M LiCl resulted in an increase in the conductivity value to 146 ± 4 mS/cm (similar to the casted materials). Both the printed and casted gels could be stretched up to 4 times their own length (after soaking). This paper contributes to the development of conducting flexible hydrogel materials.

5. Acknowledgements:

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6. References:


Preparation and characterization of a living hydrogel

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Keywords: Hydrogel, Living hydrogel, Algae, Chlorella Vulgaris, Alginate, gelatin

Recently, hydrogels have attracted great interest due to their unique material properties, which have found potential applications in a range of disciplines such as membrane electrode assemblies, fuel cells, wound dressings, neural interfaces and other biomedical applications. In this study, living hydrogels have been prepared by embedding green algae “Chlorella Vulgaris” within a hydrogel consisting of alginate. We have focused on using a sustainable, robust and abundant microorganism (Chlorella). We demonstrate that the algae cells were capable of growing within alginate hydrogels for more than 28 days. In addition, it was observed that the use of an alginate-gelatine mixture established a more robust gel system (114% increases in strength/modulus compared to alginate) after 28 days without affecting the growth of algae. These living (algae-hydrogel) materials are an important stepping stone towards the future development of living electrodes for biomedical and tissue engineering applications.

1. Introduction

Hydrogels have recently been an area of extensive focus in research fields due to their beneficial characteristics including higher water absorption capacity, hydrophilicity, biocompatibility and low toxicity [1, 2]. Hydrogels have been utilized in a wide range of biomedical uses, pharmaceutical and tissue engineering applications [3, 4]. The unique structure of hydrogels is based on a three-dimensional hydrophilic polymer network which can hold large quantities of interstitial water (usually more than 90% of their total weight) [5, 6]. Hydrogels can be prepared from various sources including natural, synthetic/natural hybrid, and synthetic polymers [7]. In recent decades, hydrogels have gained particular interest in cell culturing environments as they can mimic the entire complexity of natural tissues which are found in vivo and hence can be an ideal material for immobilisation of living cells such as microalgae [8]. Microalgae serve as an important raw material for vitamins and amino acids along with playing an important role in various fields of biomedical applications.

Moreover, it has become an important component in formation of dyes, aquaculture, animal feed and cosmetics [9]. Some of these microalgae species are used in food supplements, which include Chlorella Vulgaris and Spirulina platens [10]. Chlorella possess immune-modulating and anticancer properties, garnering much attention in the medical arena. In addition to this, Chlorella has recently been engineered to produce biodiesel, giving an alternative source of petrol from our current dependence on...
fossil fuels [11]. They play a significant role in moulding the atmospheric conditions, as they are responsible for releasing a colossal amount of oxygen, and absorbing CO$_2$ as feed via photosynthesis [12, 13]. However, the survival of microalgae species, such as *Chlorella*, in liquid media exhibit several hindrances to growth, such as potential toxicity of reactants and products, difficulties in governing the cell density, nutritional limitations and end-product recovery [14]. One approach towards solving these issues lies in the immobilisation of cells within a hydrogel. Previous studies have shown that alginate gels demonstrate promising outcomes for keeping the cells intact [15]. Alginate and gelatin are common hydrogels widely used in a variety of industries due to their biocompatibility, low toxicity level, low cost and ease of gelation [16, [1]]. However, a common issue with these hydrogels is their inherent low mechanical strength. Inter-penetrating network (IPN) systems have emerged which attempt to address this issue. IPNs are synthesized by the enmeshing of two independently polymerized networks, which can exhibit stiffer and tougher mechanical properties with controllable physical properties [18, 19].

This work will describe a simple and replicable method for cultivating algae “*Chlorella Vulgaris*” in growth media. Furthermore, immobilized algae specimens were prepared within the gels consisting of an alginate network ionically crosslinked with CaCl$_2$. Finally, mechanical characteristics were investigated to show how the growth of algae influences the gel’s physical properties.

2. Materials and methods

2.1. Materials

Green algae-*Chlorella* and its culture medium (CM3) were purchased from Southern Biological, Australia. Sodium alginate (brown algae, medium viscosity), gelatin (porcine, type A), calcium chloride dihydrate and sodium bicarbonate were purchased from Sigma Aldrich, Australia. Milli-Q water (resistivity, 18.2 MΩ cm) was used, unless otherwise stated.

2.2. Methods

2.2.1. Culturing green algae - Chlorella

Preparation of samples was carried out as follows: first 10 mL of *Chlorella* was added to 30 mL of *Chlorella* culture medium, CM3 (ratio 1:3) in a 500 ml sterile conical flask. Then, 50 mL Milli-Q water and 10 drops of 2% sodium bicarbonate was added to this solution. The flask was covered with foil to allow gas exchange while preventing bacterial entry. This mixture was cultivated under light simulated illumination at 20 µmol m$^{-2}$s$^{-1}$ with a light to dark photoperiod of 10 - 14 hours at room temperature (21 ± 2°C). Light stimulation was performed on a flask shaker (100 rpm) (Bioline orbital shaker BL8136, Australia). Algae solutions were left to culture for 28 days at 21°C.

2.2.2. Alginate and alginate-gelatine gel-forming solutions

Alginate (4% w/v) gel-forming solution was prepared by dissolving 4 g sodium alginate in 100 mL Milli-Q water under rapid stirring (1000 rpm) and heating (60°C) for 2 hours. Once fully dissolved, 4 g gelatin was added to the alginate solution and dissolved under the same conditions until a homogenous alginate-gelatin solution was resultant.

2.2.3. Hydrogel encapsulation of algae

The algae/alginate and algae/alginate-gelatin gels are constructed through mixing 3 mL of pre-prepared cultivated algae cells with 30 mL of the gel-forming solutions. They were poured into different types of sterile mould according to their subsequent characterisation method. Samples were then immersed in a
bath of 0.05 M CaCl₂ solution for 5 hours under controlled conditions (21°C; relative humidity, 45%) in order for ionic cross-linking to occur. These algae/gels were placed into sterile plastic containers containing 50 ml Milli-Q water and 10 ml of culture medium. Simulated illumination was carried out with a light to a dark photoperiod using a flask shaker.

2.3. Characterization the growth of algae

Three physical characterization methods were employed to investigate the growth of algae in media including observing the development of green colour for the solution, using a hemocytometer to monitor algae cell density (Olympus BX61; Tokyo, Japan) and assessing the algae density by spectrophotometry at 684 nm with a spectrophotometer (Spectronic 200; Thermo Fisher Scientific, India). In addition, two methods were used to track the algae growth within gels: a dissolved oxygen meter (HORIBA - OM-71, Japan) that was used to record the values of oxygen release from the algae, and an inverted fluorescence microscope to estimate the vitality of immobilized green algae in the gels. Mechanical compression testing of hydrogel materials was performed using a universal mechanical analyzer (EZ-S, Shimadzu, Japan, compression rate 1 mm/min, 50N load cell).

3. Results and discussion

3.1. Cultivation of algae in culture media

In order to evaluate the growth of algae in both culture media and hydrogels, various characterization methods were used, as described in (section 2.3). Light microscopy allowed for the observation that a green colour developed in the solution during the algae growth process (Fig. 1a-d). Moreover, the growth of algae was confirmed spectrophotometrically, where the algae exhibited an absorbance peak at 684 nm, which is characteristic to chlorophyll content. An increase in light absorbed at this wavelength can clearly be seen over time in Fig. 1e. The highest cell density and absorbance were obtained after 28 days of cultivation, which were 2860 ± 8 cells and 1.72 ± 0.06, respectively. While, as expected, the lowest cell density and absorbance were achieved at 1 day of cultivation, which were 84 ± 6 cells and 0.14 ± 0.04, respectively. This demonstrates that the algae were able to maintain a high rate of cell proliferation throughout the duration of the 28 day study under the experimental conditions.

Figure 1 Typical optical microscopy images of the increase in algae cells over time in growth media: a) 1 day, b) 7 days, c) 14 days and d) 28 days. e) Summary of algae growth in growth medium CM3, measured at 684 nm for 28 days. Insert in a)-d) show images of sterile culture flasks.
3.2. Preparation of living hydrogels

Living hydrogels were prepared using algae cells which were entrapped in two types of hydrogel materials. These gels were compared in terms of their mechanical performance along with the growth of algae cells. The results showed that the algae cells were capable of growing within either alginate or alginate-gelatin scaffolds (Fig. 2). In addition, photosynthesis activity analysis was used to determine the oxygen release in the media from the embedded algae within the hydrogel (Table 1).

A steady initial increase of dissolved oxygen ratio from algae cells which are immobilized within the hydrogel samples was observed; 10.5 ± 0.4 mg/L after 7 days compared to 7.5 ± 0.1 mg/L after 1 day. The dissolved oxygen values of the algae/alginate gels were slightly higher than the algae/alginate-gelatin through the cultivation period, as shown above in Table 1. This is because alginate scaffolds have larger pore sizes which can allow diffusion of nutrients and gases (O$_2$ and CO$_2$) more easily inside the gel network, maintaining cell viability.

Figure 2 Typical optical microscopy images show the algae growth in alginate after 14 days of incubation.

<table>
<thead>
<tr>
<th>DO ratio (mg/L)</th>
<th>Time (day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Aalgae/alginate</td>
<td>7.56 ± 0.10</td>
</tr>
<tr>
<td>Algae/alginate-gelatin</td>
<td>7.51 ± 0.13</td>
</tr>
</tbody>
</table>

Table 1 Dissolved oxygen (DO) release as a function of the growth of algae within gel scaffolds.

3.3. Characterization of living hydrogel materials

Mechanical compression testing showed that the strength/modulus of hydrogel systems decreased with increasing algae cultivation time (Table 2). The values for compressive stress at failure of algae/alginate gel decrease over time, from 145 ± 6 kPa after 1 day to 4.1 ± 0.8 kPa after 28 days. Similarly, the mechanical strength of algae/alginate-gelatin mixture decreased over time but to a lesser extent than the alginate hydrogels, with a strength of 155 ± 10 kPa at the first day and 8.7 ± 0.6 kPa after 28 days of cultivation. These results indicate that the mechanical robustness of alginate and alginate-gelatin scaffolds decreases over time. In both cases, this could attributed to either these scaffolds losing calcium ions through diffusion of the ions into the media, or by increasing algae cell numbers entrapped within these hydrogel networks.

The alginate-gelatin mixture demonstrated a more robust gel system compared to the alginate network, which is most likely a result of the formation of an interpenetrating network (IPN), which are known to exhibit better mechanical properties via synergistic strengthening [19]. Therefore, the combination of gelatin and alginate could be used to create a more stable hydrogel scaffold; however, this results in a
sacrifice of algae growth. Consequently, this work can be used to tailor scaffolds for applications that require either a robust hydrogel encapsulation or a hydrogel optimised for algae growth.

<table>
<thead>
<tr>
<th>Gel + 0.05M CaCl₂</th>
<th>Time (day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Algae/alginate</td>
<td>σc (kPa)</td>
</tr>
<tr>
<td></td>
<td>144.86 ± 6.23</td>
</tr>
<tr>
<td></td>
<td>17.76 ± 0.30</td>
</tr>
<tr>
<td></td>
<td>7.39 ± 0.76</td>
</tr>
<tr>
<td></td>
<td>6.01 ± 1.41</td>
</tr>
<tr>
<td></td>
<td>4.08 ± 0.76</td>
</tr>
</tbody>
</table>

Table 2 Mechanical properties for living hydrogels (alginate and alginate-gelatin) at 21°C. σc and Ec indicate compressive stress at failure and compression modulus (20%-30%), respectively.

Conclusions
The main objective of this work was to find a simple method for cultivating algae cells within gels that based on embedding *Chlorella Vulgaris* within alginate and alginate-gelatin mixtures. The outcomes showed that the growth of algae cells, which are immobilized in an alginate scaffold, was better than the growth of cells within alginate-gelatin mixture. However, mechanical characterization revealed that the strength/modulus of algae/alginate-gelatin gels was slightly higher (7%) on the 1st day of cultivation, and then much stronger after 28 days (114%) when compared to the equivalent algae/alginate network. Living hydrogel materials could be declared as a suitable substance for biomedical applications, especially for making living electrodes.

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References


Evaluating individual research studies using statistical techniques

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**Keywords:** Meta-analysis, grain zinc, grain iron.

Assessment is a tool that can inform decisions on whether a model is suitable for an evaluation and the type of evaluation that would be most feasible, credible, and useful. There are various methods used for estimation and evaluation in statistics such as meta-analysis. Meta-analysis allows a thematic appraisal of evidence, which can lead to a resolution of suspicions and disagreements. This study focuses on estimating the effects of CO\(_2\), water and nitrogen on grain zinc and grain iron using meta-analysis. The performances of the proposed methods are evaluated using various measurements, such as the Cochran's Q statistic and its \(p\)-value, \(I^2\) statistic, and \(\tau^2\) tau-squared. The results indicated that the zinc concentration was decreased by 0.48% under elevated carbon dioxide, dry water and low nitrogen. In addition, iron concentration was reduced by 1.67% under elevated carbon dioxide, wet water and low nitrogen. Furthermore, iron concentration was declined by 0.51% under elevated carbon dioxide, dry water and low nitrogen. The outcomes of this research will inform expert about the impact of CO\(_2\), water and nitrogen on grain zinc and iron, and allow the investigation of proper solutions.

1. **Introduction**

Statistics is a branch of mathematics that deals with the collection, analysis, testing, measuring, interpretation, presentation, and organisation of data [3]. Statistics can be applied in all aspects of life. Assessment is a tool that can inform decisions on whether a model is suitable for an evaluation and the type of evaluation that would be most feasible, credible, and useful. There are various methods used for estimation and evaluation in statistics, meta-analysis is one of them. It is a set of statistical methods for combining results from different studies into a pooled estimate of the effect size [1]. In recent years, many publications were reported to analyse the effects of CO\(_2\), water and nitrogen on crops using various methods. Of those, statistical methods were identified as important tools to study the influences of environmental factors on crops and to investigate fundamental issues concerning nutrients [12]. However, the performance evaluations were mainly conducted through traditional statistical measures such as standard deviations, variance, simple regression, ANOVA analysis of variance, t-test, \(\chi^2\) chi-square, \(R^2\) coefficient of determination, were used to assess the majority of research results [6, 11, 13]. In order to overcome this limitation, many researchers used meta-analysis as a powerful tool to investigate the homogeneities among the studies being conducted [7, 9, 10]. To our knowledge, there are no studies conducted to assess the effect of CO\(_2\), water and nitrogen supply zinc and iron levels in grains using meta-analysis. In this research, we used meta-analysis to determine the effect of CO\(_2\), water and nitrogen on grain zinc and grain iron because generalising the results from a meta-analysis makes more sense than from single studies, considering it integrates different sets of populations into the analysis.
2. Database

Different types of crops such as wheat, soybean, maize, sorghum, rice, and field peas were grown in free-air CO₂ enrichment (FACE) technology experiments with double designs, such that samples were grown at ambient CO₂ approximately 380 ppm could be compared with samples grown under identical conditions except that CO₂ was elevated to approximately 550 ppm during daylight hours. This data, collected across three continents United States America, Australia and Japan represents over ten times more data on the nutrient content of crops grown in FACE experiments than was previously available. The datasets are publicly available online at HTTP://WWW.NATURE.COM/ARTICALESSDATA201536#DATA-RECORDS [2]. As the first step in our project, we used data from Australia because it contains all information relevant with this research such as country of study, replicate (sample size), sowing timing qualitative, water qualitative, cultivar, nitrogen application qualitative, year, crop and consternation of zinc and iron. This revealed the possibility of using meta-analysis. In addition, we chose wheat yield among many crops because it does not contain missing data like others crops. Type of data: Primary Data-Raw from the studies and this data extracted from the literature. Environmental factors that used in this study: CO₂, water and Nitrogen, level of CO₂: ambient and elevated, water saturation: wet and dry, Nitrogen level: low and medium, type of crop: wheat, country of the collected data: Australia, data collection location: Victoria. The sample size of each experiment: four. The years that the experiments were done: 2007, 2008 and 2009. The concentrations of grain zinc and iron.

3. Meta-analysis Models

In meta-analysis, the effect size is estimated depending on the type of outcomes variable. There are two types of outcome variables, binary outcomes and quantitative results. The binary outcome variables include odd ratios, risk ratios and risk differences, while the quantitative outcome variables standardised mean difference (SMD), weighted mean difference (WMD) and correlations coefficient [1, 8]. There are many models in meta-analysis such as models incorporating only study effects, models incorporating additional information. The most common examples incorporating only the study effects include the fixed effects model, random effects model [1, 4, 5].

3.1 The Fixed Effect Model

The fixed effect model provides a weighted average of a series of study estimates. The inverse of the estimates’ variance is commonly used as the study weight. As a result, larger studies tend to contribute more to the weighted average than smaller studies. Consequently, when studies within a meta-analysis are dominated by a very large study, the findings from smaller studies are practically ignored [1, 8]. This is given by:

\[ T_i = \mu + u_i. \]  

(1)

where \( T_i \) is an observed effect in the study of \( i \),

\( \mu \) is common effect,

\( u_i \) is within-study error.

The weight assigned to each study is defined as:
where \( v_i \) is within study variance for study \( i \).

Then the weighted mean \( \bar{T} \) can be computed as:

\[
\bar{T} = \frac{\sum_{i=1}^{k} w_i}{\sum_{i=1}^{k} w_i},
\]

(3)

The variance of the combined effect is defined as:

\[
V_c = \frac{1}{\sum_{i=1}^{k} w_i},
\]

(4)

The standard error of the combined effect is:

\[
SE(\bar{T}) = \sqrt{V_c}.
\]

(5)

The 95% confidence interval for the combined effect is computed as:

- Lower Limit = \( \bar{T} - 1.96 \times SE(\bar{T}) \),
- Upper Limit = \( \bar{T} + 1.96 \times SE(\bar{T}) \).

(6) (7)

The Z-value can be computed using:

\[
Z = \frac{\bar{T}}{SE(\bar{T})}.
\]

(8)

For a one-tailed test, the p-value is given by:

\[
p = 1 - \varphi(|Z|),
\]

(9)

For a two-tailed test, it is given by:

\[
p = 2[1 - \varphi(|Z|)]
\]

(10)

where \( \varphi \) is the standard normal cumulative distribution function.

### 3.2 The Fixed Effect Model

A common model used to synthesise heterogeneous research is the random effects model of meta-analysis. This is simply the weighted average of the effect sizes of a group of studies. The random effects model can be written as [1]:

\[
T_i = \theta_i + e_i = \mu + \varepsilon_i + e_i,
\]

(11)

Where \( T_i \) is the observed effect in study \( i \),

- \( \theta_i \) is a true effect,
- \( e_i \) is the within-study error,
- \( \mu \) is the mean of all the true effects, and
- \( e_i \) is the between study error.
The weight assigned to each study is:

$$w_i^* = \frac{1}{v_i^*},$$  \hspace{1cm} (12)

where \(v_i^*\) is the within-study variance for study \(i\) plus the between-studies variance.

The weighted mean \(\bar{T}^*\) is then computed as:

$$\bar{T}^* = \frac{\sum_{i=1}^{k} w_i^* T_i}{\sum_{i=1}^{k} w_i^*},$$  \hspace{1cm} (13)

The variance of the combined effect is defined as:

$$V^* = \frac{1}{\sum_{i=1}^{k} w_i^*}.$$  \hspace{1cm} (14)

The standard error of the combined effect is:

$$SE = (\bar{T}^*) = \sqrt{V^*}.$$  \hspace{1cm} (15)

The 95% confidence interval for the combined effect can be computed as:

$$\text{Lower Limit}^* = \bar{T}^* - 1.96 \times SE(\bar{T}^*),$$

$$\text{Upper Limit}^* = \bar{T}^* + 1.96 \times SE(\bar{T}^*).$$  \hspace{1cm} (16) \hspace{1cm} (17)

The Z-value could be computed using:

$$Z^* = \frac{\bar{T}^*}{SE(\bar{T}^*)}.$$  \hspace{1cm} (18)

The one-tailed \(p\)-value is given by:

$$p^* = 1 - \varphi(|Z^*|),$$  \hspace{1cm} (19)

The two-tailed \(p\)-value by:

$$p^* = 2[1 - \varphi(|Z^*|)],$$  \hspace{1cm} (20)

where \(\varphi\) is the standard normal cumulative distribution function.

4. Results

Meta-analysis was carried out using the standardized mean difference (SMD) and the mean difference (MD) for continuous outcome measures (mean and standard deviation). We applied a random effects model and a fixed effect model using the inverse variance weighted approach to combine the data. \(Q\) Statistic, tau-squared and \(I\)-squared statistic were used to assess the heterogeneity among the studies [4]. Forest plots were used to interpret the statistics. To test the hypothesis of the equality of variance effect sizes, the proposal reports the value of the testing statistics and associated \(p\)-values for the different study variables. All the estimates were calculated using a computer software written in \(R\), version 3.2.5 (2016), and all the plots were calculated using the “metafor”, “meta”, “nmeta” packages, URL http://cran.r-project.org. We did eight experiments under ambient and elevate \(CO_2\) with different levels of water and nitrogen. Figure 1 represents one of the experiments that done. The results in Figure 1 summarise the effect size. \(p\)-value shows that there is no significant difference between the groups.
There are no differences in the concentration of Zn in wheat under the two groups. No heterogeneity was indicated by $Q$, $I^2$ and $\hat{\tau}^2$.

Figure 1. Is the forest plot for Zn under elevated CO$_2$ and ambient CO$_2$, the level of water is dry and the level of nitrogen is medium

<table>
<thead>
<tr>
<th>Study</th>
<th>Experimental</th>
<th>Control</th>
<th>Mean difference</th>
<th>MD</th>
<th>95% CI</th>
<th>W(fixed)</th>
<th>W(random)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>32.9750</td>
<td>6.01904</td>
<td>-1.56 [-7.50; 10.70]</td>
<td>12.2%</td>
<td>14.0%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>26.6425</td>
<td>6.08606</td>
<td>-7.89 [-14.84; -0.88]</td>
<td>21.2%</td>
<td>20.1%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>32.2975</td>
<td>6.31102</td>
<td>1.44 [-0.79; 3.62]</td>
<td>15.0%</td>
<td>16.7%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>25.1580</td>
<td>7.45809</td>
<td>-1.13 [-14.00; 4.62]</td>
<td>8.1%</td>
<td>12.1%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>27.3275</td>
<td>6.22943</td>
<td>-0.00 [-10.50; 0.50]</td>
<td>33.7%</td>
<td>24.8%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>28.2375</td>
<td>4.57115</td>
<td>-4.69 [-7.88; -1.50]</td>
<td>100%</td>
<td>—</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 1. Is the forest plot for Zn under elevated CO$_2$ and ambient CO$_2$, the level of water is dry and the level of nitrogen is medium

Table 1. Summary statistics of the pooled data

<table>
<thead>
<tr>
<th>Experiments</th>
<th>MD</th>
<th>SMD</th>
<th>P-value</th>
<th>$\tau^2$</th>
<th>$I^2$</th>
<th>$Q$</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.78</td>
<td>-0.28</td>
<td>0.87</td>
<td>(Fixed)</td>
<td>0.0128</td>
<td>2.2%</td>
<td>42.26</td>
</tr>
<tr>
<td></td>
<td>-1.63</td>
<td>-0.28</td>
<td>0.09</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.4310</td>
</tr>
<tr>
<td>2</td>
<td>-3.75</td>
<td>-0.48</td>
<td>0.0062</td>
<td>(Fixed)</td>
<td>0.0391</td>
<td>6.1%</td>
<td>20.23</td>
</tr>
<tr>
<td></td>
<td>-4.11</td>
<td>-0.48</td>
<td>0.0069</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.3809</td>
</tr>
<tr>
<td>3</td>
<td>-2.03</td>
<td>-0.48</td>
<td>0.119</td>
<td>(Fixed)</td>
<td>0</td>
<td>0</td>
<td>3.73</td>
</tr>
<tr>
<td></td>
<td>-2.70</td>
<td>-0.48</td>
<td>0.119</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.5885</td>
</tr>
<tr>
<td>4</td>
<td>-4.69</td>
<td>-0.57</td>
<td>0.073</td>
<td>(Fixed)</td>
<td>0.0345</td>
<td>5.3%</td>
<td>5.28</td>
</tr>
<tr>
<td></td>
<td>-4.68</td>
<td>-0.58</td>
<td>0.078</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.3824</td>
</tr>
<tr>
<td>5</td>
<td>-1.67</td>
<td>-0.45</td>
<td>0.007</td>
<td>(Fixed)</td>
<td>0.355</td>
<td>23.9%</td>
<td>27.60</td>
</tr>
<tr>
<td></td>
<td>-1.49</td>
<td>-0.46</td>
<td>0.01</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.1518</td>
</tr>
<tr>
<td>6</td>
<td>-2.14</td>
<td>-0.51</td>
<td>0.0031</td>
<td>(Fixed)</td>
<td>0</td>
<td>0</td>
<td>15.94</td>
</tr>
<tr>
<td></td>
<td>-2.24</td>
<td>-0.51</td>
<td>0.0031</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.6616</td>
</tr>
<tr>
<td>7</td>
<td>-1.07</td>
<td>-0.57</td>
<td>0.103</td>
<td>(Fixed)</td>
<td>1.126</td>
<td>60.2%</td>
<td>12.57</td>
</tr>
<tr>
<td></td>
<td>-1.02</td>
<td>-0.54</td>
<td>0.341</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.0278</td>
</tr>
<tr>
<td>8</td>
<td>-0.74</td>
<td>-0.28</td>
<td>0.357</td>
<td>(Fixed)</td>
<td>0.0055</td>
<td>1%</td>
<td>5.05</td>
</tr>
<tr>
<td></td>
<td>-0.85</td>
<td>-0.28</td>
<td>0.359</td>
<td>(Random)</td>
<td></td>
<td></td>
<td>0.4100</td>
</tr>
</tbody>
</table>

Table 1. Summary statistics of the pooled data

5. Discussion

In current study, the impacts of different CO$_2$, water and nitrogen on the content of zinc and iron in wheat were examined. In the second experiment the content of zinc was decreased by 0.48% and in the experiments fifth and sixth the content of iron was reduced by 0.46%, 0.51% respectively. Though, in experiments 1, 3, 4, 7 and 8 the effect of CO$_2$ was not significant. This may have resulted from the
experiments were done in one place under same conditions. The current results clearly illustrate the negative effect of elevated CO$_2$, water and nitrogen in wheat grain quality. However, only a single crop was investigated in one place. To obtain a more comprehensive picture of expected reductions in relevant grain zinc and iron, it essential to conduct studies at various places and with further crops.

6. Conclusion
The purpose of this study was to analyse the impacts of CO$_2$, water and nitrogen on grain zinc and grain iron. The proposed techniques will enhance the accuracy of analysis. The results indicated that the zinc concentration was decreased by 0.48% under elevated carbon dioxide, dry water and low nitrogen. In addition, the iron concentration was reduced by 0.46% under elevated carbon dioxide, wet water and low nitrogen. Furthermore, the iron concentration was declined by 0.51% under elevated carbon dioxide, dry water and low nitrogen. The outcomes of this research will inform expert about the impact of CO$_2$, water and nitrogen on grain zinc and iron, and allow the investigation of proper solutions. They can also be applied to other areas of study, such as plants, forest, food webs and biomedical engineering.

References


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Simulation $\alpha$ of EEG using brain network model

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Keywords: brain network model (BNM), neural masses model, structural connectivity, connectome, computational modelling, The Virtual Brain package

In this paper, we developed a large-scale brain network model comprising of four cerebral areas in the left hemisphere, and each area is modelled as an oscillator Jansen and Rit (JR) model. Our model is based on the structural connectivity of human connectome (SC) which was a hybrid from CoCoMac neuroinformatics database and diffusion spectrum imaging (DSI.) This brain network model was designed and implemented on the neuroinformatics platform using The Virtual Brain (TVB v1.5.3). The results demonstrated that incorporating the large-scale connectivity of brain regions and neural mass of JR model can generate signals similar to the $\alpha$ oscillation in frequency range of (7-12HZ) of EEG.

1. Introduction

Electroencephalography (EEG) is the recording of the brain electrical potential from the surface of the scalp using electrodes, placed on the scalp or cerebral cortex. The EEG signals is generated by the enormous synchronous dendritic activity of pyramidal cells in the cerebral cortex, which can be decomposed into distinct bands such as: delta, 1-4 Hz; theta, 4-8Hz; alpha, 7-12Hz; beta, 12-30Hz; gamma, 30-70Hz) [17]. With the availability of detailed quantitative data of anatomical connections [6, 13, 20], a new type of network models has emerged by incorporating a biologically realistic, large-scale connectivity of brain regions into dynamic network models. Recently network-modelling studies have been used to investigate the relation between structure–functional connectivity by combining the availability of full brain maps of anatomical connections with computational models of the brain’s large-scale neural dynamics [2,3,4,5,7,8,12,14]. Other efforts for computational neural modelling have also been investigated the effect of altered brain anatomical connectivity on brain dynamics [1,9] which represented a unique tool for comprehension of brain diseases.

In this study, we used a large-scale brain network model including four cerebral areas in the left hemisphere. The model consists of the brain’s anatomical connectivity (AC) -or connectome and neural masses of JR model. This structural connectivity given by the connectivity matrix of the human connectome is used to determine the coupling strengths among four cerebral areas in the left hemisphere. The single JR model describes the activity of each cerebral areas. The newly developed model has been generated self-sustained oscillation which reflects properties of the resting state networks observed in EEG within alpha-band between 7 and 12 HZ.
2. Materials and methods

2.1. Brain network model

The model architecture of this study is based on combining both an oscillator model which accounted the interaction of neural masses in one area and large-scale anatomical structure or connectomes that consist of two matrices representing the strength and the time-delay of signal transmission between each pair of brain regions. The connectivity between those neural masses of a cortical patch is replaced by connections between the mean activity of populations. The general form of brain network model (BNM) equation describes the evolution of activity of a certain node in the network. This equation is computed by summing its intrinsic local dynamics (often described by a neural mass model), short-range input, long-range input from connected regions, external input could be the noise and stimuli [18].

According to such strengths and time-delay matrices, the long-range input is computed by summing the scaled and delayed activity of connected nodes while the short-range input does not have a delay and the signal transmission is instantaneous. The evolution equation for brain network model used in this study is similar notation in above and paper of Spiegler and Jirsa [19], where this equation was described by using delayed differential system. According to Spiegler and Jirsa [19] we described our model of brain network as follows: each area in network described a neural mass of JR model, each neural mass has two state variables, namely the mean membrane potential and mean firing rate, which can describe a set $\Phi = \{\varphi_1, \varphi_2\}$. For JR model which has m=3 different neural masses (NM), namely pyramidal cells and two for excitatory and inhibitory interneuron that, the state variables of a network of m=3 NM formed as a vector $\Psi = [\Phi_1; \Phi_2; \ldots; \Phi_m]$. To link state variables $\Psi$ among 3 neural masses either excitatory or inhibitory, the square matrix of order $\sum_{i=1}^{m} n_i$ is used, where n is number of variables of neural mass. By considering the interconnection $l=4$ of neural masses model in spatial domain, where each of them represent a single cortical areas, the state variable of the resulting brain network model formed as a vector $\Omega = [\Psi_1; \Psi_2; \ldots; \Psi_l]$ according to $W_{het}$, heterogeneous connectivity matrix. Thus, $W_{het}$ can describe connectivity between 4 elements of the network (cortical areas). By applying a temporal differential operator $P \left( \frac{d}{dt} \right)$ of a network of coupled Jansen and Rit local model, the temporal evolution equation of our brain network model described below with the S ($\Omega$) is transfer function

$$P \left( \frac{d}{dt} \right) \Omega(t) = S(V_{loc}\Omega(t)) + W_{het}S(V_{loc}\Omega(t)).$$

(1)

2.2 Brain connectivity datasets: the connectome

This human connectome used in this study is publically available in the neuroinformatics platform The Virtual Brain - it is bi-hemisphere where both left and right hemispheres consist of the 38 cortical regions (listed in Table 1). Based on the default connectome in The Virtual Brain, we built the connectivity square matrix $W_{ij}$, where $1 \leq i \leq n$ and $1 \leq j \leq n$, and n is the numbers of nodes in the network comprising 4 (lA1, lA2, ICCP1, ICCR2) left nodes. This matrix of connectivity defines the connection strengths and the time-delay of signal transmission between each pair of brain regions and weights the strength of the connection between the 4 brain areas by integer values from 0 to 3 with 0 representing the absence of a connection, 1 a weak connection, 2 a moderate and 3 a strong connection (see Fig 1). To quantify the connectivity characteristics and ensure the dissipation of activity in the

\[http://thevirtualbrain.org/tvb/zwei/brainsimulator-software\]
network, We determined the connectivity measures for 4 nodes (IA1, IA2, ICCP, ICCR) including the in- and out-degree of connectivity - i.e. the in-out degree is the number of incoming and outgoing to/from a node, respectively.

2.3 JR neural mass model of temporal dynamics of an area

For the purpose of generating the temporal dynamics of each cortical areas, we modelled each 4 cortical areas through an isolated Jansen-Rit model and set the same standard values of the parameters on all 4 cortical areas corresponding to a JR model of a cortical column with ability to oscillate activity within the alpha-band (7-12 HZ) depend on the list of the parameter values (see Table 2; Jansen-Rit, [10]). As shown in Fig 2, the model of JR for cortical column consist of three neural masses: Pyramidal neural, local excitatory and local inhibitory neurons. Of each of them has the state variables namely the mean membrane potentials $V(t)$ and the mean firing rates $m(t)$.

<table>
<thead>
<tr>
<th>Label</th>
<th>Anatomical region</th>
<th>Label</th>
<th>Anatomical region</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Primary auditory cortex</td>
<td>PFCdm</td>
<td>Dorsomedial prefrontal cortex</td>
</tr>
<tr>
<td>A2</td>
<td>Secondary auditory cortex</td>
<td>PFCm</td>
<td>Medial prefrontal cortex</td>
</tr>
<tr>
<td>Amyg</td>
<td>Amygdala</td>
<td>PFCrb</td>
<td>Orbital prefrontal cortex</td>
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<td>CCP</td>
<td>Anterior cingulate cortex</td>
<td>PFCpol</td>
<td>Pole of prefrontal cortex</td>
</tr>
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<td>Retrospenial cingulate cortex</td>
<td>PFCvl</td>
<td>Ventrolateral prefrontal cortex</td>
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<td>Subgenual cingulate cortex</td>
<td>PHC</td>
<td>Parahippocampal cortex</td>
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<td>Frontal eye field</td>
<td>MCdl</td>
<td>Dorsolateral premotor cortex</td>
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<td>Gustatory cortex</td>
<td>PMCm</td>
<td>Medial premotor cortex</td>
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<td>HC</td>
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<td>PMCvl</td>
<td>Ventrolateral premotor cortex</td>
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<td>S1</td>
<td>Primary somatosensory cortex</td>
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<td>S2</td>
<td>Secondary somatosensory cortex</td>
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<td>Primary motor area</td>
<td>TCc</td>
<td>Central temporal cortex</td>
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<td>TCI</td>
<td>Pole of temporal cortex</td>
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<td>TCS</td>
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<tr>
<td>PCs</td>
<td>Superior parietal cortex</td>
<td>TCv</td>
<td>ventral temporal cortex</td>
</tr>
<tr>
<td>PFCcl</td>
<td>Centrolateral prefrontal cortex</td>
<td>V1</td>
<td>Primary visual cortex</td>
</tr>
<tr>
<td>PFCdl</td>
<td>Dorsolateral prefrontal cortex</td>
<td>V2</td>
<td>Secondary visual cortex</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CC</td>
<td>Cingulate cortex</td>
</tr>
</tbody>
</table>

Table 1. Anatomical labels and names of cerebral areas where the bold areas are included in our model.
Each of neurons population is modelled by two operators. The first operator converts the average pulse density of action potentials into an average postsynaptic membrane potential. This impulse response for the excitatory case and the inhibitory case is given by the functions below respectively:

\[
\begin{align*}
    h_e(t) &= \begin{cases} 
        Ate^{-bt} & t \geq 0 \\
        0 & else 
    \end{cases} \\
    h_i(t) &= \begin{cases} 
        Bte^{-bt} & t \geq 0 \\
        0 & else 
    \end{cases}
\end{align*}
\]

(A) and (B) determine the maximum amplitude of the excitatory and inhibitory postsynaptic potential (EPSP, IPSP), respectively. The second operator converts the average membrane potential of a neurons population into an average pulse density of action potentials. This transformation can be described by a nonlinear sigmoid function

\[ S(V) = \frac{2e^0}{1+e^r(v_0-v(t))} \]

Here, \( e_0 \) determines the maximum firing rate of the neural population, \( v_0 \) is the value of potential of PSP for which 50% firing rate is achieved, \( r \) is the slope of the sigmoid at \( v_0 \); \( v_0 \) (either the firing threshold or the excitability of the populations). This model is formulated as set six first-order differential equations as follows:
\[ \begin{align*}
y_0(t) &= y_3(t) \\
y_3(t) &= AaSi\{y_1(t) - y_2(t)\} - 2ay_3(t) - a^2y_0(t) \\
y_1(t) &= y_4(t) \\
y_4(t) &= Aa\{p(t) + C_2Si[C_1y_0(t)]\} - 2ay_4(t) - a^2y_1(t) \\
y_2(t) &= y_5(t) \\
y_5(t) &= Bb[C_4Si[C_3y_0(t)]] - 2by_5(t) - b^2y_2(t)
\end{align*} \] (5)

Thus, the temporal differential operator is described by a second order differential operator (see Spiegler and Jirsa, 2013 with its appendix A for more details)

\[ P: D_i = \lambda^2 + 2b_i\lambda + b_i^2 \] (6)

With \( b_1 = b_2 = 1 \) and \( b_3 = 1/2 \) for Pyramidal cells \((i = 1)\) with the feed loops represented by excitatory \((i = 2)\) and inhibitory \((i = 3)\).

3. Implementation and Simulation using TVB

Our model was designed and implemented by using the basis of platform TVB with its application requirements \(^3\) including an installation guide [15]. Based on structural connectivity given by the structural connectivity matrix as shown in section of Brain connectivity datasets, we were performed the pipeline simulation (region-based) of the brain network model which brings together a neural mass model with structural data according to [15,18]. The next paragraph list the sequential steps of the simulation.

1. Bringing structural connectivity given by the connectivity matrix of the human connectome to determine the coupling strengths among 4 cerebral areas. (See section the connectome).
2. Setting long rang coupling function, it is used to join the local dynamics at distinct location over connections connectivity (i.e. it is applied to the activity propagated between regions before it enters the local dynamics equations of the model. The coupling function used in this simulation is a linear function that rescales incoming activity to a level appropriate to the population model with \( \lambda = 4.2e^{-8} \) (0.0000000042).
3. Using the conduction speed. In this study we used 4mm/ms of speed of signal propagation through the network.
4. Embedding the model for the local dynamics into network. Here we used the JR’S model with its parameters. Before embedding it in a network. We identified and tuned the parameters of the local dynamics to typically used values as given in JR model (see Table 2) where this operation of parameters tune was implemented by phase plane tool in order to explore the model and how the dynamics of this physical model change as a function of its parameter. This tool is already included in TVB.
5. After we have defined our structure and dynamics of model, the numerical integration of the system was performed using Hen’s method – is available for solving ordinary different equations (ODES) with an integration step size of \( 0.01220703125 \) ms.
6. Recording the relevant data from the simulation which is simply raw neural activity described by the state variables of JR model using the Temporal average monitor with sampling period 1 ms.
7. Finally, the simulation length was 1000 ms.

\(^3\) [http://docs.thevirtualbrain.org](http://docs.thevirtualbrain.org)
Model Parameter | Interpretation | Value
--- | --- | ---
A | Average excitatory synaptic | 3.25mV
B | Average inhibitory synaptic gain | 22mV
a | Time constant of excitatory PSP | 100s⁻¹
b | Time constant of inhibitory PSP | 50s⁻¹
C | Average number of synaptic between populations | 135
C1, C2 | Average Probability of synaptic contacts in the feedback excitatory loop | C1=C C2=0.8C
C3,C4 | Average Probability of synaptic contacts in the slow feedback inhibitory loop | C3=C4=0.25
v₀ | The value of the average membrane potential | 6Mv
Vmax | Threshold | 5s⁻¹
r | Steepness of the sigmoidal transformation | 0.56mV⁻¹
e₀ | The maximum firing rate of the neural population | 2.5 s⁻¹

Table 2. Standard Numerical Value used in Jansen’s model

Figure 2: (A) Schematic connectivity of the Jansen & Rit model of a cortical column. Pyramidal cells (PC) receive excitatory and inhibitory feedback loops from local interneurons represented in populations of excitatory and inhibitory interneurons (EIN, IIN, respectively) and excitatory input from neighboring or more distant columns, and it give excitatory input to EIN and IIN. The arrows of excitatory and inhibitory connection marked with ‘+’, ‘−’, respectively. The four connectivity constants C₁,...,C₄ represent the strength of the connections. (B) Block diagram of JR model which represents the mathematical operations performed inside a cortical area. The postsynaptic boxes labelled hₑ(t) and hᵢ(t) in the figure correspond to linear synaptic integrations while the boxes labelled Si represent the cell bodies of neurons and correspond to the sigmoidal transformation that convert the membrane potential of a neural population into an output firing rate. The constants Cₙ model the strength of the synaptic connections between populations and y₀,y₁,y₂ are three main variables in the model as the main outputs of the 3 postsynaptic boxes[10].
4. Results

Figure 3. Simulated alpha activity. (A) Time series of each area constituted the output of the postsynaptic potential (PSP) noted $y_0$ (see figure 2B) of each areas in the network in the left hemisphere. Top: IA1 and IA2. Bottom: ICCR and ICCP. (B) Time series show the simulated alpha activity for 4 left interconnected cerebral areas in the network.

Because left hemisphere appears to dominate the functions of speech, language processing and comprehension, and logical reasoning, more recently, human studies have been reported that alpha-band oscillations reflect one of the most basic cognitive processes [11] such as attention, perception, memory, language, learning, and higher reasoning and it also finds enormous interest as a potential biomarker for disorder or disease as well as playing a key role in understanding other EEG phenomena due outstanding features of the alpha frequency band [16]. In this study, we have focused on this kind of oscillation generated in the left hemisphere by integrating global dynamics with a local model (mesoscopic) that determines the dynamics within brain regions. By running the Simulation steps used TVB in section above, the results of model of this study showed that the simulation of the large-scale brain network model based on Jansen-Rit population, enables to produce a signal similar to spontaneous EEG alpha oscillation within frequency range of (7-12HZ). Our finding for our large-scale brain network model depend on Jansen-Rit model represented the time series of the mean PSP of the PC noted $y_0$, 
in figure 2b of each area. This time series constituted the output of each area in the network where each area instantaneously transmitted the mean firing rate of its PC to other linked areas as shown in Figure 3.

5. Discussions

We have presented a brain network model at the meso-and macroscopical scale of neural populations used here a simple neural mass—the JR model. The result from our model represented the time series of the mean postsynaptic potentials of the PC for 4 left interconnected cerebral areas in the network. We think of this quantity of y1 as the important output also as noted in figure 2b because in the cortex the PC are the main vectors of long-range cortical connections and also their electrical activity corresponds to the EEG signal. So we further simulate the mean EPSP of the PC noted y1 in figure 2b for four areas as shown in figure 4.

Figure 4. Time series show the simulated alpha activity for 4 left interconnected cerebral areas in the network constituted the output of the mean EPSP of the PC of each areas in the network noted y1 in figure 2b.

6. Conclusion

The focus of this study was to generate a signal similar to spontaneous EEG alpha oscillation within frequency range of (7-12Hz) using the brain network model including four cerebral areas in left hemispheres, each area is modelled by a Jansen and Rit model which accounted the interaction of neural masses in one area. The structural data used in this work is a human connectome. This model was developed and implemented on the neuroinformatics platform of The Virtual Brain (TVB v1.5.3). Our the simulation results show that the model has the potential to produce the electrical brain activity due to its performance and flexibility as well as for reproducibility of the simulations. Future studies will focus on extending the model to 38 cerebral areas in left hemisphere and also to a full model with 76 cerebral areas including both of the left and right hemispheres. The proposed model has the potential to help physicians to accurately diagnose especially with lower alpha frequency ranges for cognitive and creative tasks.

References


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A Preliminary Exploration of the GitHub Ecosystem: How to find important repositories

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Keywords: Open source software, GitHub, Repository, Online communities

GitHub is arguably the most influential OSS version control system currently available. It is utilized by indie developers and large global companies. GitHub is an OSS developer service hub where a creator’s raw project data can be accessed. However, with large numbers of developers, and significant numbers of projects hosted on GitHub, it remains difficult to extract useful knowledge from such projects since not all repositories are legal, accurate, or still active. In this paper, we suggest better ways to query GitHub and extracted useful repositories that highly related to search criteria. The paper also contains a trend analysis of the top JavaScript, Java, and Python GitHub repositories. Observations indicate there are attributes other than star and fork that could be useful for better query result when searching GitHub to direct result away from questionable repositories. The paper concludes by offering insights into future larger scale studies.

1. Introduction

Open source software (OSS) is an approach used to solve a diverse range of project-specific software problems. In recent years, GitHub has become the world’s largest code host collection of OSS development projects, with more than 5M developers collaborating across 10M of its project repositories [1]. This paper considers key constructs affecting the successful development of GitHub OSS repository projects, and the development of successful OSS.

GitHub is an online version control system used by online open source software developers ranging from: professionals to students, from major software companies to small Indie developers [2]. As well as GitHub provides an environment for developers to share their work as well as offering it for others to use and adapt and get help improving their work [2]. GitHub works as follows: a developer creates a repository, the content of repository organised into branches, a “master” branch represents the “production code”, other branches are used for repository contributors to experiment with new features, and the restructuring existing features. Repository evolves over time from the addition and deletion of content (primarily: source code, resource files, and documentation). its changes are tracked via “commits” - a set of additions and deletions to the content. A developer can “clone” a repository - they get a complete copy of the content they can use for their own purposes. Also they can “fork” a repository to get a complete copy of the content placed in a new repository that they own, this might represent: 1) a fracture in the ecosystem; 2) a way for contributors from the original repository to work more independently/safely away from the original content - GitHub uses the Git SVN tool - the tool’s workflow is complex and error-prone; or 3) a way for a non-contributor of the original repository to make suggestions and prove their ideas are useful - they might wish to become part of the original repository.
Pull requests (a commit that is “merged” into the repository only after approval by the contributors) are the mechanism of suggesting changes/improvements for content. The originating repository development occurs via commits and branching by the repository originator and contributors who have been added, forked repository’s [3].

GitHub provides social networking tools for developers to communicate, discuss and reason about pull request. It provides many statistics to track all this information (via GitHub API). GitHub enables the creation of large complex interconnected ecosystems of developers (it is easy for developer to make a diagram, showing the possible relationships between repository’s, and collaborates with other developers) [4].

GitHub project repositories offers a trackable, integrated, time-spanned, workflow of user-delivered, project contributions. But the datamining tools used in GitHub remain lightly different from those used in other OSS developments [3]. GitHub repositories are also growing rapidly compared to other OSS communities [1]. The growth and success of GitHub OSS development communities can be viewed as a social activity across contributing developers. This creates a ‘herd behaviour,’ and with growth, project leadership becomes increasingly important. GitHub attracts software developers, testers, star coders, social media watchers, and other small solutions pull providers. Its project repositories are easily deployed, follow clear guidelines, engage suitable languages, and are readily utilized to improve the existing OSS development version [3].

For example, and active repository can be mapped against the number-of-commits (and pulls) over a time interval provided there are more than two committers into the author’s GitHub project. However, although the number of issues is also a project success indicator some active projects do not engage GitHub’s issue tracker [5].

Finding the most important / useful repositories is problematic. Most users searching GitHub based on fork counts and star counts - which by itself gives results that range from truly important projects to irrelevant results. For developers to discover repositories they are interested in they need a good searching criteria.

Many online tools exist for finding important / influential repositories. They are a small fraction of the total number of repositories on GitHub. So what if a developer needs to find repository that is important to him and it’s not on the list these tools provide.

This paper presents a pilot study of selected GitHub repositories, utilizing GitHub’s data querying API to explore OSS trends based on GitHub meta-data, and how contributors influence the development of OSS projects. Against these GitHub constructs the paper investigates:

- Any relationship between ASD and OSS development,
- Parameters that could help in mining GitHub repositories (it remains difficult to extract useful knowledge from such projects since not all repositories are legal or accurate or still active, hence guidelines are necessary to isolate those questionable repositories).
2. Methodology

GitHub is selected for this pilot case study as it represents the largest OSS indie. It uses a metadata extraction process – with extractions taken across evenly spaced time intervals. Snapshots have been taken of repositories measures per language - each set of measures occur 2 weeks apart: stars, commits, forks, issue closing, pull closing, watchers, and the rest. The authors expected that each repository would be a mature long-running project, formed a complex community of developers interested in the features and utility of the repository. Mature project as likely to be using a form of Agile software development, a typical agile development iteration lasts two weeks, so we should be capturing activity that represents work equivalent at least a single iteration.

**Dataset:** The top 10 Java Script, Java, and Python project repositories, each having significant forking numbers, were chosen as representative of each of the three mostly widely-used programming languages [6,7,8]. Using a “general search” tools in GitHub not limiting to particular type of project.

**Process:** two processes have been established: extract and analyse, in extract the authors used GitHub API to extract repositories information which were: *Repositories name, Stars, Forks, version, Contributors, Commits, addition, deletion (message changes), issues (open and close) and Pull (open and close).* Extraction is the most consuming time, it is not possible to extract all the commits and its addition/deletion in one step, loop through each message should be read to get the updated commits. API authentication access needed in this step the authentication enables 6000 API retrieves per time interval. Some repositories required more than 6000 access which enforce a waiting time to resume the extraction process according to GitHub rules. The analysis process takes place by calculating and comparing repositories meta-data for a period of 4 weeks every two weeks, thus capturing 3 data points per programming language.

**Timescale:** two weeks is the periods to redo the process, the two weeks’ philosophy followed by agile to produce repositories development. The development includes changing or updating repositories information’s (all or some of them). Accordingly, repositories that follow agile development methodology will have a lot of different activity at each time scale.

3. Pilot Study

GitHub provides a search tool to discover “trending” repositories based on the metrics “most forked” and “most starred” these metrics attempt at capturing the “value” or “importance” of a repository. It has sophisticated & complex-to-understand search options. Most people use the “basic search” - not a sophisticated search. Basic search has been used to obtain the data set.

In this study, the authors observe the 30 most popular GitHub Repositories (most forks repositories) each 10 repositories representing a programming language. The programming languages for the data sets are for JavaScript, Java, and Python. Table 1 illustrates the key GitHub data attributes that can be collected using the GitHub API.
<table>
<thead>
<tr>
<th>Statistic</th>
<th>Meaning</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stars</td>
<td>Project interest</td>
<td>Developers who like the repo</td>
</tr>
<tr>
<td>Watches</td>
<td>Project interest</td>
<td>Developers who get notification when the repo content changes</td>
</tr>
<tr>
<td>Contributors</td>
<td>Project interest</td>
<td>Developers who asked to directly contribute to a repo</td>
</tr>
<tr>
<td>Commits</td>
<td>Project work</td>
<td>Content changes resulting from new features, refactoring, and incremental development</td>
</tr>
<tr>
<td>Forks</td>
<td>Project work</td>
<td>Isolated versions of a repo where changes are made to the original content or its intent</td>
</tr>
<tr>
<td>Releases</td>
<td>Project work</td>
<td>Milestones in the lifetime of a repo</td>
</tr>
<tr>
<td>Issues open</td>
<td>Change request</td>
<td>Identified problems with repo content</td>
</tr>
<tr>
<td>Issues closed</td>
<td>Change request</td>
<td>Issues fixed by commits or merges after a review process</td>
</tr>
<tr>
<td>Pulls open</td>
<td>Change request</td>
<td>Suggested commits from forked versions of the repo or within the repo from developmental branches</td>
</tr>
<tr>
<td>Pulls closed</td>
<td>Change request</td>
<td>Pulls merged into the repo after a review process</td>
</tr>
</tbody>
</table>

Table 1: GitHub repository data attributes, their associated meaning and descriptions.

Summarization of the results for all three programming languages are illustrated in Table 2. As can be seen from this table, all values increased except for issue opened which actually decreased. Studying these repositories provides some indication about what factors affect the activity of those particular GitHub projects, and whether or not standard Agile methodology time-boxed is being practiced during the current development of these projects.

Observation shows that pull request, watchers, releases and commits are the most influencing factors affect project activity. Increasing these values will increase projects followers and participations. The commits highly influence activity levels of the repositories, number of commits increased with an increasing number of watch. As commits increased close counters (for issues and pull) increased too fig.1 below illustrate these results, as could be seen the three programming languages share the same trends. every 14 days the authors observe changing in the above factors, after 14 days the authors notice growth in most repositories, the growth oriented toward issues open or closed mostly, 5 out of 30 repositories have low activity (the only activity was responding to pull request). Increasing in commits affect the watchers, star and issues closed.
In this study, the top 10 GitHub repositories for the top three programming languages were explored (as an example of an OSS model), and observe the development of these repositories for the period of 45 days every 14 days to calculate factors that affect activity log for these repositories. Our trend analysis reveals consistent patterns across this short timeframe that imply there might exists a culture of development on GitHub.

Our future work will explore this. Results included a wide variety of project types. Some of the trending repositories, some questionable repositories, and some educational / getting started repositories. In

Figure 1. Trend analysis of GitHub attributes for three programming languages.

Table 2: Data collected for the attributes of GitHub

<table>
<thead>
<tr>
<th>Time</th>
<th>Javascript</th>
<th>Java</th>
<th>Python</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time</td>
<td>Commits</td>
<td>Committers</td>
</tr>
<tr>
<td>1Day</td>
<td></td>
<td>71843</td>
<td>2520</td>
</tr>
<tr>
<td>14Days</td>
<td></td>
<td>72692</td>
<td>3035</td>
</tr>
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<td>3046</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Time</th>
<th>Javascript</th>
<th>Java</th>
<th>Python</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td>Committers</td>
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<tr>
<td>28Days</td>
<td></td>
<td>65414</td>
<td>763</td>
</tr>
</tbody>
</table>

4. Conclusion and Future work

In this study, the top 10 GitHub repositories for the top three programming languages were explored (as an example of an OSS model), and observe the development of these repositories for the period of 45 days every 14 days to calculate factors that affect activity log for these repositories. Our trend analysis reveals consistent patterns across this short timeframe that imply there might exists a culture of development on GitHub.

Our future work will explore this. Results included a wide variety of project types. Some of the trending repositories, some questionable repositories, and some educational / getting started repositories. In
answering to the question of the relationship between ASD and OSS, OSS developers tend to “cherry-pick” practices from a variety of development methodologies. The results seem to show that the developers as working on the snapshot of repositories have similar development behaviours.

There was very little change in in the rate of all measures over each snapshot, some clear differences between repositories grouped by language. In referring to research question related. The more commits history means that repository is important and still active.

Our datasets appear to indicate that using basic search method in GitHub (depending only on number of forks) produces results that are questionable. For example, the “shadowsocks” repository which is illegal but still has large fork and star numbers, and the Heroku node JavaScript which is a sample not a complete repository. From a data mining perspective: a data scientist would consider such results “outliers” that need to be filtered [4,8].

From our study query result could be more critical when incorporating project activity factors. From Figure 1 it is obvious that the GitHub attributes are (commits, watchers, fork, stars, release and pull). Our pilot study used a small dataset to give a snapshot of the internal repositories activity. Future work will be aimed at mining bigger datasets to better understand the GitHub ecosystem on a wider range of repositories. Moreover, an investigation of the relationships between different repository features may allow us to better understand the repository activity levels in detail. Observing repositories for a long period of time is expected to allow the measurement of periodically changes of those repositories.

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References


Agronomic response of sorghum (*Sorghum bicolor* L.) to different nitrogen fertiliser rates under controlled and non-controlled traffic farming systems in red Ferrosol

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Keywords: Enhanced nitrogen formulations, Nitrogen-use efficiency, Soil compaction, Urea ammonium nitrate, Urea, Sorghum.

Compaction adversely affects the physical properties of soils and the ability of crops to efficiently use water and nutrients, and therefore reduces the amount of fertiliser recovered in grain. This study was conducted to investigate the effect of traffic compaction on sorghum response to nitrogen (N) fertilisation. Fertiliser was applied at rates between 0 (control) and 300 kg ha\textsuperscript{-1} N at regular increments of 100 kg ha\textsuperscript{-1} N using urea (46% N), urea-ammonium nitrate (UAN, solution, 32% N) and ENTEC® urea (46% N). Soil conditions (density) representative of controlled and non-controlled traffic farming systems were achieved by removing compaction through subsoiling to a depth of approximately 300mm and by performing six passes of a medium-sized tractor, respectively. The soil type used in the study was a Red Ferrosol (69% clay, 11% silt, 20% sand) and sorghum was grown during the 2015-2016 season. Grain yield was approximately 40% higher in CTF compared with non-CTF, and consistent with differences (P<0.05) in all measurements of crop yield components. Improved soil structural conditions are therefore a pre-requisite for increased fertiliser use efficiency.

1. Introduction

Controlled traffic farming (CTF) is a mechanization system in which tramlines and seed beds are distinctly and permanently separated to optimize conditions for trafficability with farm machinery as well as soil conditions for crop growth. Recent studies [1] have shown that CTF systems have the potential to either reduce nitrogen (N) fertiliser inputs without compromising crop yield or increase crop yield for the given fertiliser input. This is supported by studies showing enhanced structural conditions in soils established under CTF [2], and by enhanced nutrient uptake in the absence of traffic compaction [3]. However, no detailed studies have been reported on the effects of traffic compaction on the actual yield-to-fertiliser response relationships from which optimum economic N application rates could be derived, particularly
for subtropical edaphoclimatic conditions. Therefore, work was undertaken to determine the effect of traffic compaction on the yield-to-nitrogen response of sorghum crop.

2. Materials and Methods

2.1. Experimental site

The experiment was conducted at the Experimental Station of the University of Southern Queensland (27°36'35.27"S, 151°55'50.62"E) located in Toowoomba (Queensland, Australia) during the summer of 2015-2016. Total rainfall in May 2015 (138 mm) largely exceeded long-term (1970-2014) records for this month (57 mm), and it was relatively lower in June-July and October 2015, respectively. Overall, mean air temperatures did not depart significantly from long-term records, despite that minimum temperatures were slightly below average, particularly in early spring.

The soil at the site is described in [4] as a Red Ferrosol, which is well-drained and has a gentle slope (<0.8%), and it is similar to those frequently occurring in Queensland. Soil textural analyses [5] for the bulked 0-200 mm layer were: 69% clay, 11% silt, and 20% sand. There was a requirement to remove historical near-surface compaction at the experimental site to enable the two traffic treatments (CTF and non-CTF, respectively) to be imposed [6]. For this, the soil was first chisel-plowed to a depth of 300 mm. This cultivation depth was chosen based on an earlier study in SE Queensland [7], which showed that removal of compaction to such depth was sufficient to return mine-rehabilitated land affected by compaction to satisfactory crop production and that rainfall-use efficiency achieved after cultivation was ≥85% in most years.

Soil conditions (density and strength) representative of controlled and non-controlled traffic systems were achieved by removing compaction through subsoiling to a depth of approximately 300 mm and by performing six passes of a medium-sized tractor (Belarus 920, 100 HP, gross mass: 3.9 Mg), respectively as demonstrated in the earlier work [8]. Sorghum (Sorghum bicolor L.) was sown on 11th of November 2015 at a field-equivalent seeding rate of 2.5 kg ha⁻¹, and subject to standard agronomic practice; except for the fertiliser application, which was dependent on treatment. Sowing was conducted with a 4-row conventional driller fitted with knife points at 750 mm row spacing.

The experiment was conducted in two adjacent blocks; namely: CTF and non-CTF, in which 60 plots (dimensions: 4-m × 4-m) with 4 plant rows per plot were laid-out in a completely randomised design, and subject to the fertiliser treatments described here. Three types of fertiliser were used: urea (46% N), urea treated with 3,4-dimethyl pyrazole phosphate (DMPP), commercially known as ENTEC® urea (46% N), and urea ammonium nitrate referred to as UAN (30% N, solution). All fertiliser treatments, including controls, were setup in triplicate (n=3). The fertilisers were hand-applied in a single band (=50 mm) next to the plant row and incorporated at N rates between 0 (control) and 300 kg ha⁻¹ N at regular increments of 100 kg ha⁻¹. For all fertiliser treatments, the application of N fertiliser was split into two dressings for the rates of 200 and 300 kg ha⁻¹ N: applied at 30- November and the second dressing was applied about two weeks later at 11th of December.

2.2. Soil measurements and analyses

Soil bulk density (ρₚ) was determined for the 0-300 mm depth layer at regular increments of 100 mm by taking soil cores of 50 mm in diameter. Measurements were taken three times (n=3) before and after the traffic treatments were imposed, and ρₚ was determined based on [9] (Table 1). Cone penetrometer
resistance was measured by pushing a cone (125 mm² base area, 30° apex angle) into the soil to a depth of 500 mm at constant speed (0.05 m s⁻¹), and digitally recording the force at 25 mm depth increments based on ASABE Standard EP542 [10]. Soil moisture content was simultaneously determined because of its influence on soil strength [11]. Measurements were conducted ten times (n=10) for each traffic treatment. Soil water infiltration was measured using the double-ring infiltrometer method [12]. Infiltration rates were subsequently obtained by differentiating Kostiakov’s equation with respect to time to describe the relationship between the rate of infiltration and time. Measurements were replicated three times (n=3).

2.3. Crop measurements and analyses
The crop was harvested by hand-cutting the entire plants from entire plot at approximately 20 mm above the soil surface on 4th of March 2016. These samples were used to determine grain yield, and the following yield components: harvest index (HI), the ratio between grain yield and weight-total aboveground biomass [13]; thousand grain weight (TGW) [14]. Total N in grain [14] was used to estimate apparent N recovery in grain by the difference method, and hence N use efficiency (NUE). Differences in yield between fertilised and non-fertilised crops, relative to N applied as fertiliser were used to denote agronomic efficiency [15].

3. Results and Discussion
3.1. Soil physical properties
Soil penetration resistance determined for the CTF and non-CTF systems is shown in Fig. 1. Overall, there were significant differences (P<0.05) in soil cone index between the two traffic systems, particularly in the 50 to 200 mm depth interval, where penetration resistance was up to 65% higher in non-CTF. Mean values of cone index in the 0-500 mm depth range were 2.5 and 5.12 MPa for the CTF and non-CTF systems, respectively. Comparing the penetration resistance profiles from wheeled area with non-wheeled plots showed that the layer ranged from 0-200 mm was more affected than the rest of depths although the impacts of machinery traffic on all layers were significant. Soil moisture content was lower than field capacity (ranged: 10-16 %) at the time of measuring soil penetration resistance, this might be the reason that caused the increase of PR values under both traffic systems in this particular soil (red Ferrosol). The other reason that led to an increased PR value and relatively decreased bulk density in the clay soils was described by [16]. The researchers reported that soils with a large amount of fine particles (silt and clay) will have smaller pore diameters and a higher penetration resistance at a lower bulk density than a soil with a large amount of coarse particles.
### Table 1. Soil bulk density under CTF and non-CTF. The standard deviation (SD) is shown as ± the mean value (n = 3).

<table>
<thead>
<tr>
<th>Traffic system</th>
<th>Depth (mm)</th>
<th>Bulk density (g cm(^{-3}))</th>
<th>Total porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTF</td>
<td>0-150</td>
<td>1.22±0.06</td>
<td>54±0.02</td>
</tr>
<tr>
<td></td>
<td>150-300</td>
<td>1.20±0.03</td>
<td>55±0.02</td>
</tr>
<tr>
<td>non-CTF</td>
<td>0-150</td>
<td>1.37±0.05</td>
<td>49±0.01</td>
</tr>
<tr>
<td></td>
<td>150-300</td>
<td>1.38±0.04</td>
<td>48±0.01</td>
</tr>
</tbody>
</table>

3.2. Grain yield and harvest index

There were significant differences in grain yield and yield components between the two traffic systems as well as between fertilised (treated) and non-fertilised crop (controls), which were observed in both traffic systems (P < 0.05). Yield components were also significantly affected by traffic system and nitrogen application rate (P < 0.05).

Comparisons between controls showed that grain yield was about 480 kg ha\(^{-1}\) greater in CTF compared with non-CTF (P < 0.05). The fertilised crop under CTF system was approximately 1400 kg ha\(^{-1}\) higher compared with non-CTF (P < 0.05). The optimum N application rates (MERN), and corresponding grain yield, were 145 kg ha\(^{-1}\) N and 3428 kg ha\(^{-1}\), and 100 kg ha\(^{-1}\) N and 1796 kg ha\(^{-1}\) for CTF and non-CTF, respectively. The effect of fertiliser type on the grain yield and yield components was not significant (P > 0.05), which suggested that compaction was the main factor influencing the response to applied N fertiliser (Fig. 2 a). Thus, grain yield was relatively more sensitive to soil compaction than fertiliser N formulation. This effect was consistent at any given rate of nitrogen fertiliser.
Differences in harvest index between treatments and controls were significant in both traffic systems as well as between CTF and non-CTF (P<0.05). Harvest indices were higher when fertiliser was applied at rate of 200 kg ha$^{-1}$ N, which was in accord with estimates of optimum N application rates (Fig.2 b).

![Figure 2](image_url)

**Figure 2.** The relationships between nitrogen application rates and grain yield (A); and harvest index (B) of sorghum under CTF and non-CTF systems. Error bars denote standard deviation (SD) values at n = 9 (except n = 3 for N=0 and N=MERN).

The reduction in grain yield caused by traffic of farm machinery was due to reduced root exploration of subsurface soil layers, and limited extraction of soil nutrients and moisture [17]. Other studies had attributed the reduction in crop yield and agronomic efficiency to the crop under compacted soil (non-CTF system) to limited supply of water, oxygen and nutrients from the soil to the root system or a limited activity of the root system [18]. Similar findings were reported in the earlier study for wheat crop under...
the similar soil type and conditions [8]. Significant differences in harvest index due to fertiliser treatment and traffic system was concurrent with changes in grain yield and total aboveground biomass [8].

4. Conclusions
This study has investigated the agronomic performance of sorghum crop (Sorghum bicolor L.) grown in compacted and non-compacted soils to represent the conditions of non-CTF and CTF traffic systems, respectively. Based on the field measurements, it has been found that soil physical properties such as bulk density, penetrometer resistance and water infiltration rate were improved significantly under controlled traffic system by 12%, 56% and 50%, respectively. The grain yield and harvest index were also enhanced under the condition of CTF system by 40% and 26%, respectively.

5. Acknowledgment
This project received financial and operational support from the National Centre for Engineering in Agriculture (NCEA) at the University of Southern Queensland, (Toowoomba, Australia), and Iraqi Government represented by the Ministry of Higher Education and Scientific Research. Technical assistance provided by colleagues and staff members at USQ and CSIRO (Toowoomba, QLD) is gratefully acknowledged.

6. References


Controlled traffic farming reduces energy requirements in Australian cropping systems

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Keywords: Controlled traffic farming, Draft, Soil compaction, Soil engaging implements, Motion resistance.

Soil compaction due to non-controlled traffic (non-CTF) of agricultural equipment can reduce crop yields and increase the cost of energy use on-farm. Traffic compaction in non-CTF systems can cover up to 80%, and commonly 50% of the planted area each time a crop is produced. Field access, particularly under wet conditions, can also be restricted because of typically low soil bearing strengths, which reduces the ability of soil to support traffic. The impact of preceding wheel traffic on tine draft and energy requirements of soil-engaging operations during planting in two contrasting soil types (heavy clay and loam) were investigated. Results showed that the motion resistance of machinery was between 30% and 45% lower in CTF compared to non-CTF depending upon the soil type. The draft of seeder opener tines positioned behind a tractor wheel increased by 30% when operated on the heavy clay soil, and by 15% when operated on the loam soil. These results clearly showed the potential of CTF to significantly reduce energy requirements of seeding.

1. Introduction

Soil compaction is a phenomenon that affects agricultural lands throughout the world. It reduces water and air entry into surface and subsoils, increases resistance to root penetration with associated problems of increased runoff and erosion, and consequently reduces root growth and crop yields [1]. Compaction of soil by farm machinery traffic is a major problem of agricultural production. The soil subjected to traffic could be compacted to ~80% of the maximum compactability depending on soil type and its conditions at the time of traffic, and the characteristics of the equipment used [2]. Non-controlled traffic farming (non-CTF) can cover about 50% crop area in no till systems, and >80% of area each time a crop is produced [3]. This compaction due to non-controlled traffic (non-CTF) of these farm machinery can increase the cost of energy use on-farm and reduce yield the most agronomic crops [4-5]. The cost of soil compaction is estimated in Australia as equivalent to agricultural production loss of approximately AUD850 million per year [6]. Many techniques and farming systems are being developed to eliminate, alleviate and manage soil compaction in field such as: reducing pressure on soil either by increase contact area or reduce axle load; working soil at optimal soil moisture (below soil moisture deficit (SMD)) ; reducing the number of passes; increasing soil organic matter; mechanical loosening of soil (deep ripping); crop rotations that include plants with deep, strong taproots (biological loosening of soil) [7, 8]. These approaches cannot achieve their full potential with non-CTF (random trafficking). The compaction resulting from machinery traffic in a field is usually unseen, as there may be no visible evidence of the problem on the soil surface [9]. In recent years, the Agricultural machinery has become progressively...
more powerful and efficient but heavier, with increasing impacts on soil [10-11]. For example, an axle weight of 10 tonne or more will cause severe compaction to depths of 300mm or more. Avoidance is the best strategy for managing compaction [12], but without CTF it would seem unlikely that any modern broadacre cropping system would be immune from the impacts of compaction[13]. Controlled traffic farming is an approach not only to manage compaction in a field but also to utilize it to provide the strong soil needed by agricultural tyres for efficient traffic and traction [14]. In this regard, the separation of compacted traffic lanes and non-compacted cropping areas in CTF can improve energy use efficiency through all of these avenues [15]. The Australian Controlled Traffic Farming Association Inc. [16] defines CTF as a system in which: (1) All machinery has the same or modular working and track gauge width, which enables establishment of permanent traffic lanes, (2) all machinery is capable of precise guidance along those permanent traffic lanes, and (3). Farm, paddock and permanent traffic lane layout are arranged to optimise drainage and logistics. A number of authors have investigated the effect of CTF on crop yield. [17] found that sugarcane productivity and sugar yield increased by 18.72% and 20.29%, respectively, as a result of using management systems with traffic controlled. [18] also found that CTF has a significant impact on yield of wheat where the replicated plot experiments in non-traffic soil demonstrated a significant differential of +16% compared to random traffic. CTF farmers have provided many anecdotal reports of reduced power requirements and fuel consumption, so the energy effects of CTF are clearly important, but published evidence is not unanimous. [19] working on a clay soil in Australia observed that the traffic effect of wheels on the draft of tillage implements increased total draft by 30% or more compared with the same implement operated in non-trafficked soil. [20] in the USA and [21] in UK however, found that traffic systems had no significant effect on energy requirements, but these studies considered only the draft force effects, not motion resistance. It also appears that there are still no studies of CTF energy effects which include motion resistance effects. The objective of this study is thus a more complete assessment of CTF effects on the energy requirements of extensive grain cropping systems in Australia. It is important to note that these are predominantly “no-till”, with no soil disturbance prior to the seeding operation.

2. Material and methods

2.1. Tine draft measuring unit and pull meter

The unit used to measure draft forces is shown in Figure 1. Draft-sensing was achieved with chisel plough shanks attached to parallel link assemblies, movement of which was restricted by shear beam force transducers (SKT model 1500). These were monitored by a data logger (Rimik DataNode) providing an oversampling and decimation system for filtering signal noise. The mean draught force for each transducer was logged at two-second intervals. The four-parallel link assemblies were mounted on a 4-m wide three-point linkage toolbar fitted with adjustable depth control wheels at its extremities (Figure 1), and fitted with common proprietary shares: chisels, sweeps and seeding openers.
Pull meter was used to measure the force due to the Motion resistance. The capacity of the load cell is up to 35 kN, this was monitored by a data logger (Rimik DataNode), work principle of this data logger was similar to that of data logger of tine draft measuring unit.

**Experimental design and sites description**

The draft force experiments were carried out using a 550-m × 15-m plot arranged in a complete randomized block design, with three replications. Three factors and different levels of comparison, namely: soil conditions (Non-wheeled to represent CTF, wheeled to address non-CTF), working depth (50-75, 75-100, and 100-125 mm, respectively) and type of tine (seeder opener) were used. The working depths were chosen to represent those commonly used for seeding, fertiliser application, and deep placement of fertilizer, respectively. The narrow points (seeder openers) were used for planting and fertilizing. A working speed of 8 km.h⁻¹ was used during the tests treatments. Motion resistance experiments were also arranged in a complete randomized block design, with three replications. Two factors were used: soil conditions (tramlines to represent CTF, permanent crop beds to address non-CTF, and on a hard surface (e.g., dirt track or road). This latter measurement was used to assess internal friction and tyre deflection. All measurements were undertaken at a common ground speed of 8 km.h⁻¹. The ground speed was chosen to represent that commonly used for seeding operating. During the tests treatments were repeated into opposite to reduce the effect of soil topography. This requirement affected the layout of the motion resistance experiments, a pull meter, which connected between two tractors, was used to record the motion resistance.

Experimental sites were established on commercial farms at Pittsworth (Qld) 27°, 45’, 4603 South latitude, +151°, 27’.7265 East longitude, and Swan Hill (Vic) 34°, 55’, 8049 South latitude, 143°, 2°, 6826 East longitude, representing a range of no-till grain production systems. They were operated under permanent CTF. Soil types were a Gray Vertisol (=60% clay), and Red Sodosol (loam), respectively. Grain crop such as wheat, barley, canola and a variety of pulses, were the main crops on the farms depending on season.

**Soil physical properties**

Soil properties measured included bulk density, soil moisture content, and soil shear, the parameters considered most likely to influence energy consumption during field machinery operations. Bulk density was assessed using the core method [22]. Measurements of bulk density were carried out in permanent traffic lanes (tramlines) and permanent crop beds at soil depths; 0-50; 50-100; 100-150 mm, covering the entire working depth range. Measurements were also replicated and randomised within plots to decrease the effect of soil heterogeneity. A shear vane device was used to assess this in-situ. Soil shear
force is expected to relate to motion resistance, and also provide information indicative of implement performance [23]. Gravimetric soil moisture was determined by weighing and oven drying method as described by [24] The samples were taken for bulk density measurement were also used to determine the moisture content to cover the entire work depth.

Method of data analysis
Statistical analyses were conducted using Statistical Package for Social Sciences (SPSS). Analysis of variance (ANOVA) was performed using 1% and 5% probability levels, and least significant differences were used to compare means.

3. Results and discussions
3.1. Soil physical properties
There were significant differences of soil properties between permanent traffic lanes and permanent crop beds for tested soil (p-values <0.001) (Table 1). Bulk density measured in tramlines and permanent crop bed increased with increasing soil depth, which was observed for both soils (Table 1). Significant differences in bulk density were observed on tramlines compared with permanent crop bed as shown in Table 1. The permanent traffic lanes (tramlines) showed higher bulk density compared with permanent crop beds 1.674 and 1.249 Mg. m\(^{-3}\), respectively. These observations were for both soils. The highest bulk density was observed on tramlines at loam soil (1.748 Mg. m\(^{-3}\)). The least values were obtained on permanent crop beds at clay soil (1.175 Mg. m\(^{-3}\)). Similarly, shear force showed significant differences between tramlines and permanent crop bed in both soils. The shear force was lower for permanent crop beds compared with tramlines (0.328 and 0.074 MPa, respectively). There was some missing data in Table 1 because the soil was hard to press the shear vane device in permanent traffic lanes at Sawn Hill site in particular. The moisture content exhibited higher values in clay soil than loam soil at both permanent crop beds and permanent traffic lanes in particular (Table 1). These results demonstrated that controlled traffic has the potential to reduce soil bulk density and shear force in permanent crop beds at both sites. However, at permanent traffic lanes significantly increased bulk density shear force in both tested soil. These results were in agreement with [25] and [26], which indicated that isolation of traffic in no-till systems was efficient in improving soil physical conditions. These results had a positive significant effect on energy requirements.

<table>
<thead>
<tr>
<th>Pittsworth site</th>
<th>Soil properties</th>
<th>Bulk density (Mg.m(^{-3}))</th>
<th>Shear force (MPa)</th>
<th>Soil Moisture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth (mm)</td>
<td>Mean</td>
<td>Standard deviation</td>
<td>Mean</td>
</tr>
<tr>
<td>Permanent traffic lanes (tramlines)</td>
<td>50</td>
<td>1.543 **</td>
<td>± 0.002</td>
<td>0.251**</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>1.654 **</td>
<td>± 0.027</td>
<td>0.328</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>1.607</td>
<td>± 0.049</td>
<td>0.340**</td>
</tr>
<tr>
<td>Permanent crop beds (CB)</td>
<td>50</td>
<td>1.005 **</td>
<td>± 0.105</td>
<td>0.051**</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>1.227 **</td>
<td>± 0.060</td>
<td>0.074</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>1.294</td>
<td>± 0.047</td>
<td>0.181**</td>
</tr>
</tbody>
</table>
Table 1. Bulk density and shear force of soils sites

<table>
<thead>
<tr>
<th></th>
<th>Swan Hill</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Permanent traffic lanes</strong></td>
<td></td>
</tr>
<tr>
<td>(tramlines)</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>150</td>
</tr>
<tr>
<td><strong>Permanent crop beds</strong></td>
<td></td>
</tr>
<tr>
<td>(CB)</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>150</td>
</tr>
</tbody>
</table>

** Significant

3.2. Energy requirement

Draft force of seeder openers at different depths were significantly different (p<0.001), and increased significantly as the work depth increased (Figure 2). The draft force was greater in wheeled than non-wheeled 2.620 kN and 1.907 kN at Pittsworth site, and 1.344 kN and 1.186 kN at Swan Hill site, respectively. The difference between draft in wheeled and non-wheeled soil found in this study was similar, but they were smaller than those reported in earlier work [19]. These results also show that even one pass by tractors with weight of 112 kN and 174 kN at Pittsworth and Swan Hill, respectively could cause significant damage. In this regard, up to 80% compaction damage occurs in the first pass [27]. These results clearly show that CTF reduced draft force of seeder openers at seeding and fertiliser application by up to 30%, at Pittsworth and up to 15% at Swan Hill site. These differences in result between Pittsworth and Swan Hill sites were affected by soil moisture and soil texture where Pittsworth site had a higher soil moisture compared to Swan Hill site. In addition, soil texture was loamy in Swan Hill as opposed to clayey in Pittsworth. Soil of Pittsworth site was more sensitive to compaction than the soil of Swan Hill site. This was in agreement with [28, 29], which indicated that soil compaction caused traffic or others (natural factors), is related to soil properties such as soil texture, clay mineralogy, organic matter content sesquioxide content, exchangeable cations, and soil water content.

Figure 2. Tractor wheel traffic effects on the mean draft force of seeder openers at different depths in clay soil at Pittsworth QLD (left), and in loam at Swan Hill VIC (right). A vertical bar is LSD.

Statistical analysis of the motion resistance data indicated that this was also significantly affected by soil condition (p<0.001). Motion resistance was always less when a tractor was driven in an existing wheel-track, and increased significantly when the tractor was driven off the wheel-track, and onto non-wheeled soil (Figure 3). The motion resistance effect is much more significant in the loam soil of Swan Hill site compare to clay soil of Pittsworth site where observed the reduction of motion resistance by 45% and 30% repressively. Obvious soil compaction was observed in the surface layer of the track (Table 1), this soil compaction in the vehicle tracks can improve traffic conditions and increase the traction efficiency.
knowing that bearing capacity of firm soil is high enough to support the equipment [14][30]. Motion resistance is a much more important component of overall energy requirements in no-till systems, which have steadily displaced tillage-based systems in Australia and large parts of the world. In no-tillage systems, the sprayer and spreader was intensively used to control the weed in the field, which reflected in the cost of weed management including fuel consumption. CTF can reduce the cost of fuel consumption by decreased motion resistance for farm equipment. CTF can also assessed no-tillage system by reducing the motion resistance in particular.

![Percentage reduction in motion resistance](image)

**Figure 3.** Percentage reduction in motion resistance achieved by driving a tractor in existing wheel tracks in clay soil at Pittsworth (QLD) and loam at Swan Hill (VIC). A vertical bar is LSD.

### 4. Conclusion

The present study has investigated the effects of CTF on the energy requirements of extensive grain cropping systems in Queensland and Victoria, and shown that CTF systems had a significant impact on soil properties. Draft forces in both CTF sites were significantly lower than in non-CTF. CTF reduced energy required for seeder openers by 30% in heavy clay soils and by 15% in medium-textured soils. CTF reduced motion resistance of farm equipment by ≈30% in heavy clay soils, and by 45% in medium texture. The above results could translate into 20-25% reduction in fuel use.

### 5. Acknowledgements:

The authors are grateful to Iraqi government, the Australian Controlled Traffic Farming Association Inc., the Grains Research and Development Corporation (Australian Government), and USQ for financial and operational support and Dr Dio Antille for technical discussions. Access to their fields and technical assistant from growers in southern Queensland and Victoria is gratefully acknowledged too.

### 6. References


Investigating the validity of the Godwin & O’Dogherty single tine model for a red Ferrosol clay soil - Queensland

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Keywords: Draught force, predicted draft force, measured draft force, single tine model.

Draught force is an important parameter for measuring and evaluating implement performance to determine energy requirements per hectare. Tool geometry, operating- and soil-conditions are considered the basis of any tillage implement draught force. Prediction of tillage operation draught requirements is to allow optimisation of tine design and provide cost estimates of in field operations, which may allow very deep tillage for compaction removal. The Godwin and O’Dogherty model is the most widely accepted model for predicting theoretical tillage draught force. The main advantage of the Godwin and O’Dogherty model is in its simplicity. However, it has not been validated for the Australian soils that typically have very high clay content within the tillage depth. This study investigated the validation of the Godwin and O’Dogherty model in a ferrosol clay soil (≈70% clay). The result showed that the Godwin and O’Dogherty model can predict the draft force for a single tine deep ripper (subsoiler) with an average error of -12.92 % and up to +3.79 %, compared with measured draft force. Therefore, there is merit in further testing the Godwin and O’Dogherty model for Vertisol soils with clay content between 50–90%.

1. Introduction

Draught force is an important parameter for measuring and evaluating implement performance to determine energy requirements per hectare. The draft is defined by [1] as “the horizontal component of pull, parallel to the direction of travel, imposed on the tractor by the implement being pulled”. Tillage equipment is considered one of the primary implement to creating the desired seedbed with the purpose of increasing crop yield [2]. Recently, there has been a trend to increase the ploughs size to meet the growing demand for food as a result of the growing population growth [3]. Moreover, tillage equipment companies tended to overdesign their products as a result of the lack of proper design and appropriate analytical model to optimise the strength of frame of the tillage implement [4]. Thus, increasing pull requirements and loads on both soil and tractor which in turn leads to increased fuel consumption, reducing the machinery unit performance efficiency and creating soil compaction. Tool geometry, operating- and soil-conditions are considered the basis of any tillage implement draft force [5]. Therefore, one of the aims of tillage equipment design is reducing tool pull forces especially in heavy clay soils [6]. Prediction of tillage operation draft requirements will allow tine design optimisation and provide cost estimates of in field operations, which may allow very deep tillage for compaction removal [7]. In compacted soil, the cone index usually ranges between 2 to 3 MPa [8] while [9] concluded that the cone index ranges between 2 - 2.5 MPa for compacted soil. With the beginning of the 1960s, several mathematical models have been developed by a number of researchers e.g. [10], [11], [12], and [13] for predicting the tillage tool performance characteristics in different soils [14]. Recently, [15] have described the development of tillage tools prediction models from simple plane tines to mouldboard
ploughs in relation to tool geometry and soil physical properties. Theoretically, Godwin and O’Dogherty draft force equation considered a Universal Earthmoving Equation (Eq.1) [16] and the most widely accepted analytical model [1]. In addition to the multiple advantages of their model, they prepared a number of spreadsheets which covered single and interacting tines, discs, anchors, and mouldboard ploughs to facilitate the measurement, calculation, and prediction of draft force. The input parameters of Godwin and O’Dogherty single tine model are divided into soil and tine parameters; bulk unit weight; cohesion; internal friction angle; surcharge; soil-metal friction angle are soil parameters while depth; width; rake angle; and velocity are the tine parameters. The draft and vertical force are the output parameters. [17] said that increasing working depth from 0.38 to 0.46 m lead to decreasing the practical speed from 0.94 to 0.89 m/s respectively to keep the required power at about 32kW. The internal and external friction angle, cohesion and adhesion of soil have an impact effect on the draft force. According to [18], the internal friction angle ranges from 18˚-50˚ for sand and from 0 -37˚ for clay. The spreadsheets guidelines of [15] showed that the moist clay cohesions ranges from 10 -40 kN/m$^2$ and can rise to 100 kN/m$^2$ for hard dry soil. Moreover, they mentioned that for dense wet and clayey soil, the bulk unit weight reach to 18 kN/m$^3$ and the adhesion should be measured while it can be considered negligible in all cases. [15] concluded that their model has the ability to predict the draft force within bounds of ±20%. Finally, the main advantage of Godwin and O’Dogherty model is in its simplicity. However, it has not been validated for the Australian soils that typically have very high clay content within the tillage depth [19]. In this study, the applicability of the Godwin and O’Dogherty single tine model has been investigated by comparing predictions draught force results with experimental results in a red ferrosol heavy clayey soil (=70% clay) under two different working depth.

2. Material and Methods

2.1. The Godwin and O’Dogherty model input and output parameters

According to [16], most of the analytical predictions of draft forces acting on wide tines are based on the equation of [15] (Eq. 1):

$$ P = \left( \gamma d^2 N_\gamma + cdN_c + C_a dN_c a + q dN_q + \frac{v^2}{g} dN_a \right) w $$  \hspace{1cm} (1)

Where: $P$ is the draught force, $\gamma$ is the soil bulk unit weight, $d$ is the depth, $C$ is cohesion, $C_a$ is adhesion, $q$ is surcharge pressure, $g$ is the gravitational acceleration, $V$ is the working velocity, $W$ is the width, and $N_\gamma, N_c, N_ca, N_q, N_a$ are dimensionless factors.

2.2. Trial site and draught force components

The field trials were conducted during 2017 on red ferrosol clayey soil [20] at the University of Southern Queensland (Ag plot) (27° 36′ 35. 93” S, 151° 55′ 49.43” E) (Fig.1) with a soil profile of 69.06 % clay, 10 % silt, and 20.94 % sand to compute the draught force under two different tillage depths (0.25 - 0.3 and 0.55 - 0.6 m). A horizontal and bonded strain gauge (pull meter) (Fig.2-D) which has an overall draft load capacity of 30 tons (= 300 kN) was connected on one side to the drawbar of ahead tractor ((John Deere 6150 M (engine: 116.3 kW, PTO: 91.7kW, overall weight: 5929 kg)) (Fig.2-A) and in other side to the neutral tractor drawbar (Belarus 920 (engine: 74.6 kW, PTO: 68.6 kW, overall weight: 3900 kg)) (Fig.2-B) to measure draft force. The force was monitored, collected, and saved by a data logger (Rimik digital Data-Node) (Fig.3-C) which wired connected to the pull meter and located inside the ahead tractor cab. The average draft force for measurements was recorded at 2-second intervals. Prior to the test, the pull
meter was calibrated with tension device in the USQ, Faculty of health, Engineering and Science, Z1 Lab. Single tine ripper (Barrow) with a 0.095 m wide foot (tip), 0.04 m wide straight shank, 35° rake angle and 0.7 maximum depth (Fig.2-C) was connected to the neutral tractor via three-point linkage.

**Figure 1.** Experiment field in University of Southern Queensland (27° 36’ 35.93” S, 151° 55’ 49.43” E)

**Figure 2.** Draught force components (tractors, pull meter, and the single tine ripper)

**Figure 3.** Rimik CP40ii Cone Penetrometer, ShearTrac-II device, Post Driver, and the pull meter
2.3. Soil measurement instrumentation

For soil dry density and moisture content, a portable Christie’s Engineering CHPD78 Post Driver, Australia, which equipped with Honda GX35 4-stroke motor was used to push steel sampling tube into required depth (Fig.3-D). A mounted Rimik CP4011 Cone Penetrometer, Australia, with a 100 kg load cell on constant drive device was used to measure the soil penetration resistance (Fig.3-A). A strain-controlled direct shear test equipment (ShearTrac-II), Geocomp Corporation, USA, with up to 4.4 kN load capacity has been used to determine the cohesion, adhesion, internal, and external friction angle (Fig.3-B). A number of steel rings with a circular iron block, which fit with the diameter, and volume of direct shear device box have used to provide undisturbed soil samples from the specific operation depth.

2.4. Field Application of Methodology

Firstly, a 100 m field length was determined and flagged. Before the trials, the ripper connected to the neutral tractor (Belarus 920) via three-point linkage and the operation depths level (0.25 - 0.3; and 0.55 - 0.60 m) was hydraulically regulated through number of passes. After that, the pull meter connected between the two tractors. A 2.7 km/h was elected as a theoretical speed through operation of the ahead tractor (John Deere 6150 M). The two tractors with pull meter and the ripper moved through the 100 m distance at the respective treatment depths. In this case, the pull meter results represented both measured draft force and the rolling resistance. Next, rolling resistance of the neutral tractor was obtained while the ripper lowered to nearly touching the ground. The net measured draft force for ripper pulling was then calculated as the difference between the measured draft force and the rolling resistance value. In addition, the time taken by the machinery during the 100 m distance was recorded to determine the practical speeds. At the end of any tillage run, while the ripper still penetrates the soil, the soil in front of the tip was shovelled to measure the rake angle. The Rimik Cone Penetrometer was used to measure soil penetration resistance every 10 m, penetration resistance was measured every 0.025 m to an extent of 0.75 m, resulting in 330 measurements per 100 m distance (Fig.4).

![Figure 4. Dry density, cone index, Bulk unit weight, and soil moisture of experimental site](image)

Not far from the penetration locations, stainless sampling tube was pushed via Post Driver to a depth of 0.8m to coincide with the maximum depth of cone penetration resistance. The soil samples had very
carefully been split into 0.1m subsamples which resulted in 88 samples. Subsamples were stored in sealed foil-lined bags and weighed as soon as possible after sampling to determine the field wet weight of each sample. Subsamples were dried to constant weight at 105°C and for at least 72 hours to determine the oven dry weight. Dry density was then calculated by the ratio of subsamples oven-dry mass to its volume while sample moisture content was determined by difference of the wet and oven dry weights [21] (Fig. 4).

Steel rings have used to provide undisturbed soil samples from the specific operation depth with a circular iron block for measuring the cohesion, adhesion, internal, and external friction angle. Five normal loads (100.53, 157.19, 314.16, 471.24, and 628.32 N) were applied during the test and the shear force applied with a constant driving mechanism and recorded when the soil sheared for each normal force applied. The normal stress and the shear stress were given automatically through the facility of the (ShearTrac-II) device. The procedure was derived according to the ShearTrac-II instruction manual.

3. Result and discussion

The investigation results showed that the mean required draft force of single tine ripper (Barrow) increased from 8.59 to 41.70 kN and the practical speed decreased from 0.76 m/s to 0.67 m/s when the operation depth increase from 0.25m to 0.6m respectively and this agrees with [17]. Therefore, the drawbar power ranged from 6.44 kW to 27.8 kW when operating at 0.25 to 0.6 m respectively. From feeding the model with the input parameters (Table 1), the mean predicted draft force ranged from 7.48 kN to 43.28 kN when the working depth increased from 0.25 to 0.6 m respectively.

The results of ShearTrac-II device (Table 1) were for undisturbed soil samples which were taken from compacted horizons 0.2 – 0.3 m (3.9 MPa) for deep one (0.25m) and from 0.5 to 0.6 m (4.6 MPa) for deep two (0.6m) and they agree with [15, 18]. [15] have concluded that their model has the ability to predict the draft force within bounds of ±20%. The comparison between the measured and predicted draft force has shown that the Godwin and O’Dogherty single tine model has predicted the draft force with an average error ranges from -12.92% (7.48 kN) to +3.79 (43.28 kN). Accordingly, the model considered valid during this investigation work.

<table>
<thead>
<tr>
<th>Parameters symbol</th>
<th>Definition</th>
<th>Working depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>w</td>
<td>Width of the subsoiler foot (tip)</td>
<td>0.095m</td>
</tr>
<tr>
<td>α</td>
<td>Rake angle</td>
<td>35°</td>
</tr>
<tr>
<td>v</td>
<td>Forward velocity</td>
<td>0.76 m/s</td>
</tr>
<tr>
<td>γ</td>
<td>Bulk unit weight</td>
<td>17.84 kN/m³</td>
</tr>
<tr>
<td>C</td>
<td>Cohesion</td>
<td>32.48 kN/m²</td>
</tr>
<tr>
<td>Cₐ</td>
<td>Adhesion</td>
<td>18.5 kN/m²</td>
</tr>
<tr>
<td>φ</td>
<td>Internal friction angle</td>
<td>36.84°</td>
</tr>
<tr>
<td>δ</td>
<td>Soil-metal friction angle</td>
<td>23.2°</td>
</tr>
</tbody>
</table>

Table 1. The input parameters of Godwin and O’Dogherty single tine model during the experiment.
4. Conclusion

Draught force is an important parameter for measuring and evaluating plowing implement performance to determine energy requirements per hectare. Prediction of tillage operation draught requirements will allow reducing the draft force through optimisation of tine design. Godwin and O’Dogherty are one of many researchers who develop models to predict the draft force for tillage tools. Despite the good advantages of their models, they have not been validated in Australia soils. In this study, Godwin and O’Dogherty single tine model has been investigated in red ferrosol clayey soil. The investigation results have shown that the Godwin and O’Dogherty single tine model had predicted the draft force with an average error ranges from -12.92 % (7.48 kN) for the first working depth (0.25m) to +3.79 % (43.28 kN) for the second working depth (0.6m) in comparison to the measured draft force results (8.59 and 41.70 kN) respectively. Thus as a consequence of this work, the model is considered validated Therefore, there is merit in further testing the Godwin and O’Dogherty model for Vertisol soils with clay content between 50–90%.

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References


BUSINESS

An Investigation of Leadership Styles and Strategic Planning Processes in the Success of Public and Private Colleges in Baghdad, Iraq

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Keywords: Balanced Scorecard, Higher education, Leadership styles, Multifactor Leadership Questionnaire, Organisational Success.

This research will explore current approaches to strategic planning and the styles of leadership in use in a random sample of government and private colleges of higher education in Baghdad. In gaining insights into organizational culture and factors that impact on success, the outcomes are expected to contribute to government policy change that will ultimately enable the adoption of strategies for improvement. A mixed methods approach will be adopted where data will be initially collected using three questionnaires to managers and leaders. These will be the Multifactor Leadership Questionnaire, the Balanced Scorecard and Survey of Organizational Culture. The second stage will involve interviews with a sub-sample of these participants. Analyses of these will provide descriptive statistics on the nature of the colleges’ current practices and perceived challenges, and inferential statistics to identify the relationships and effects amongst the variables of leadership style and organizational success. The final stage will share the research findings and recommendations in focus-group discussions to finalize. The research outcomes have the potential to impact on all colleges in facilitating centralized policy change and help ensure they are in the best position to develop and adapt to better compete and grow, despite the ongoing turbulent environment.

1. Introduction

The higher education sector in Iraq in post-war 2003 has the potential to be more effective in contributing to the country’s rebuilding and revival. The role of higher education in development through increasing the number of colleges for both the public and private sector has been emphasized in recent times [1]. However, international events have created many challenges for both public and private colleges in the world [2], and correspondingly, in Iraq. For example, Iraqi colleges have faced the same external challenges such as decreasing government funding, increasing demand for higher education, changes in student demographics, economic transformation, the challenges presented by competing organizations, and also the increased demand to teach Western knowledge [3,4]. Similarly, these colleges have faced internal challenges such as poor leadership style and communication among members, unwillingness by leaders to listen to staff comments and suggestions, a lack of acknowledgment of staffs’ work and poor organizational outcomes [5,6]. Hence, the question arises as to how these colleges currently address and face these challenges, and in turn the way their leaders are
approaching the situation. There is a need to investigate, since Haber-Curran have shown that to deal with such challenges and achieve an organization’s goals and objectives needs transformative and adaptive leadership styles [7]. Similarly, it has also been confirmed that organizational success depends on good leadership and the type of leadership employed [8]. Thus, in keeping with the findings of these studies the proposed research is justified in its aim to investigate more deeply the relationship between leadership styles and organizational success, since their practices are not clear and their outcomes vary [9].

1.1. Research Aim, Objectives and Questions
The aim of this research is to explore the leadership styles (LS) evident among leaders and managers of a random sample of government and private colleges of higher education in Baghdad in relation to their strategic planning processes and organizational success (OS) to identify strategies for how their organizational outcomes might be improved. The research questions are as follows:

1. How is leadership manifested in the sample colleges in terms of styles, organizational culture, strategic planning and organizational success.
2. What are the challenges that leaders and managers currently face in their need to improve their organizations success and why?
3. How do the colleges currently plan to address the challenges in their particular context and provide leadership to achieve their current goals?
4. What is the relationship between their LS and OS in relation to the colleges outcomes?

1.2. Significance of the research
The research is very important for two major reasons. Firstly, this research focuses on the educational service sector in Baghdad, Iraq, which plays a crucial role to improve the level of knowledge and skills of students in the various disciplines and the colleges’ ability to develop good practice and adapt to the changing environment and achieve/improve success. Secondly, the research is empirical research in its aim to examine the effect and relationship between leadership styles and organizational success, for which there is a dearth of research in relation to higher education in the Iraqi context.

2. Literature Review
2.1. Research Conceptual Framework
The research’s conceptual framework draws upon a critical review of the literature, which shows that according to current research findings leadership is correlated with and acts as an independent variable with OS being the dependent variable. In other words, OS depends on the effectiveness of LS in operation but since the effect and relationship of LS to OS is not clear, this research seeks to illuminate this aspect of the colleges’ operations in relation to their achievement of OS, thus addressing the current gap in knowledge.

Figure 1 outlines the research conceptual framework, showing the two variables of LS and OS and depicting the related aspects of the Balanced Scorecard that contribute to the way they may impact on OS. These include the four elements of and their use in the way their data may be used to support leadership. The research will adopt four basic stages to research how organizational success might be best achieved, taking into account leadership styles and the way the colleges attempt to achieve success.
In applying the Balanced Scorecard it will explore the colleges’ practices and outcomes in relation to Internal Processes, Finance, Customer Satisfaction, and Learning and Growth through perceptions of staff and any supporting documentation.

Figure 1. The research conceptual framework
Source: Author 1

This will provide insights into perceived effectiveness and efficiency, level of satisfaction of students, employees, and shareholders, and through the financial perspective insights into prosperity and viability, as well as the extent the colleges are perceived as learning organizations. Therefore, the research’s conceptual framework will work as system theory where inputs, processes, and outputs can be considered along with feedback and learning. The Balanced Scorecard will provide a framework for investigating each college’s organizational success based on the data gathered for its four areas shown in blue.

2.2 Organizational Leadership
In the consideration of organizational leadership, it is necessary to distinguish between Leadership, Leader, and Leadership Style as follows:

- Leadership is “a group function: it occurs only when two or more people interact”
- Leader” intentionally seeks to influence the behaviour of other people”
- Leadership style is “characterised by the leader adopting the role of mentor, adviser and strategic planner” [9]. The leader is seen as being involved in motivating workers, providing them with an inspirational vision and purpose in their work, encouraging positive attitudes and promoting the values [10,11, 12]. In the literature on leadership there are numerous typologies of leadership [12, 13, 14], and in recent times adaptive leadership has also been recognized [15]. In the Iraqi context, transformative style is identified as an important focus in the research [16, 17], but evidence of transactional leadership style has also been found [3, 6]. Whereas, other studies have highlighted the importance of researching adaptive leadership style particularly in relation to the Iraqi higher education sector as [18]. Therefore, the research remains open in its consideration of leadership styles not discounting the fact that leaders may employ more than one leadership style in order to respond to their organization’s context [19]. It is noted that adaptive leadership style relates well to transformational leadership as it involves the ability to mobilize people to tackle challenges, and grow and change, and in enabling the capacity for an organization to thrive in new and different environments, and incorporate new dreams, new strategies and abilities [20].

2.3 Organizational Success
In this research, it is necessary to address what is meant by organisational success as being seen as influenced by leadership style. Thus, the organizational success might be defined, "effectiveness and
efficiency” [5]. However, the research of long-term success indicators applicable to competitive markets may also be a consideration, because these indicators can deliver data to benchmark and compare an organization’s outcomes with its competitors in the same field. Some researchers and experts claim this as core to understanding organizational success as [21,22]. Other researchers indicated that the application of the ‘Balanced Scorecard’ is a significant stage in planning to ensure the achievement of organizational success. It is seen as having a critical impact in strategy formulation and implementation [23,24].

In view of this, the researcher defines organizational success as the organization’s ability to manage its human capital, knowledge, and strategies in a way that competitors cannot easily imitate, or learn from its experiences. The adoption of the Balanced Scorecard approach [23,25] is further justified because it is well recognized in performance evaluation by the Ministry of Iraqi Higher Education and Scientific Research. In addition, it is concerned about satisfaction of the beneficiaries of education besides human capital, profitability, productivity, market share (reputation), public liability, and creativity as essential indicators to measure each college’s performance and success.

3. Methodology
3.1. Research Philosophy and Design
A mixed method approach was selected for this research as it was viewed the most appropriate approach to gain an in-depth understanding of the current situation of the colleges or educational organizations in the context of this study [16]. This design will enable the researcher to explore the current approach to and relationship between leadership and organizational success in the selected colleges [1].

Figure 2. Data and information collection process of Stage 1
Source: Author 1

3.2. Sampling and Unit of Analysis (Participants)
The researcher adopted the purposive sample method to select the participants (158) who comprise the leaders and managers (deans, dean assistants and members of the colleges’ council) in a randomly selected sample of six public and six private colleges in Baghdad (27%). This selection is justified on three grounds. Firstly, the management level of colleges has the authority of making strategic decisions. Secondly, they are well experienced in the Iraqi higher education sector and have a high level of knowledge, skills and experience with respect to the work in their faculties. Finally, they are the main actors that make decisions related to their colleges and have the governance to take them.

3.3. Data Collection Instruments and Issues of validity and reliability
The data collection includes both quantitative and qualitative data in the administration of the Multifactor Leadership Questionnaire (MLQ) [26] and survey of Organizational Culture [26], in addition to the Balanced Scorecard, followed by interviews with a subsample of participants to follow through on the emergent issues and finally focused group discussions on the research findings and recommendations. The literature review will also include governmental documents and reports, and websites. The Multifactor Leadership Questionnaire is already internationally validated and is provided
in the Arabic language. The additional survey will be trialed and tested for reliability using Cronbach’s Alpha [27]. It will be piloted by number of college staff who represent the membership of the selected research sample but from other colleges. The trial will enable a check on the appropriateness of items, the effectiveness of the questionnaire, to make sure that the potential participants can complete the questionnaire without any misunderstanding.

4 Outcomes and Expected Contribution to Knowledge

1. This research will make contribution to knowledge regarding how the strategic planning process, college leadership, and organizational success relate together, and how strategic planning can best be applied in higher education organisations in Baghdad and in turn Iraq since colleges across the country follow the policy and procedures developed centrally.

2. This research may enable and ensure that the Iraqi colleges can develop their practice and adapt to compete and grow despite the ongoing turbulent environment through the project’s recommendations.

3. It will seek to address the challenges and illuminate the journey of colleges’ improvement to identify common issues and the way leadership style combined with strategic planning processes impact on success to achieve a re-conceptualised view to support colleges’ improvement in current challenging times.

5. Summary

Most of the colleges in the world and particularly in the Iraqi colleges there is a need for strong and committed leadership to face the variety of challenges involved as noted earlier, including the economic and financial, and the demands for increased opportunity in higher education, changing student demographics, and competition between the emerging private and public colleges in the need for restoration and capacity building. The research also responds to the claims by some studies that some colleges in higher education fail because of weak leadership, which has led to a drop in the colleges’ success [17, 28, 29].

Consequently, these colleges need to research the relationship and effect between leadership styles and organizational success to enable them to provide their services and better respond to the changing needs of their customers.

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6. References


