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Determination of Reinforced Concrete Rectangular Sections Having Plastic Moments Equal to all IPE Profiles

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Received 02 January 2021; Revised 10 March 2021; Accepted 19 March 2021; Published 01 April 2021

Abstract

The comparison between steel structures and reinforced concrete structures has always been governed by economy and response to earthquake. Steel structures being lighter and are thus more efficient to resist earthquake. On the other hand, they are more expensive (4 to 5 times). Theoretically, two structural elements having the same plastic moment have an equal failure or collapse load. Different profiles of IPE are realized in industry and all their characteristics are determined with a great precision (weight, geometrical characteristics and thus their plastic moment). Determining equivalent rectangular singly reinforced concrete cross-sections is not easy and seems impossible to be solved analytically. To a given profile it may be found a multitude of equivalent rectangular reinforced concrete cross-section (singly and doubly reinforced with different yield strengths and compositions of concrete). To take into consideration all these factors, it is absolutely necessary to construct three axis design charts with an appropriate choice of system of coordinates in order to cover all possible ranges of different parameters. The choice of all these possible rectangular reinforced concrete sections is governed by the plastic performance of these later. They must be under reinforced, allowing plastification of steel before failure in order to permit the redistribution phenomenon in plastic analysis. The exploitation of these different charts has revealed that the absolute majority of these rectangular reinforced concrete cross-section are reasonably well designed and are in conformity with the dimensions used in practice. The results of the present characterization using Eurocode 2 characteristics are compared to those of CP110. The impact does not seem to be very relevant.

Keywords: Reinforced Concrete; IPE Profile; Equivalent Sections; Plastic Moments; Reduced Moments; Reinforcement Ratio.

1. Introduction

The comparison between steel and reinforced concrete structures has been the subject of several studies which have generally focused on the economic aspect and seismic behaviour as well as the durability and strength of each material [1, 2]. Steel structures being lighter and thus more efficient to resist to earthquakes, but on the other hand they are more expensive (4 to 5 times).

All structural elements made of steel or reinforced concrete follow the same load transfer law (material strength), but they differ in several aspects, such as the material itself and its behaviour, plastic bending resistance capacity (loads and failure modes), stability and durability, etc. However, each type of structure has advantages and disadvantages. The dimensioning of the cross-sections of these structural elements is directly affected by the safety margin adopted by the different codes, and therefore the change of code has been the subject of several comparative

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http://dx.doi.org/10.28991/cej-2021-03091677

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Knowledge of the real plastic capacity in bending of structural elements and identification of their collapse loads and failure modes play an important role in rational dimensioning [5, 6]. Ashrafzadeh and Kheyrolahi [7], studied the behaviour of folding steel structures by a durability method, where structural performance under seismic excitation was among the main objectives of their research. Al-Ansari and Afzal [8], presented a simple method for estimating the flexural design strength of reinforced concrete beam sections with irregular shapes. The method was based on structural safety and reliability.

Experiments for relevant loading rates and pressures reveal that steel and concrete exhibit complex non-linear behaviour that is difficult to capture in a single constitutive model [9]. The flexural capacity and ductility of reinforced concrete beams with several usual and non-usual geometrical shapes have been estimated theoretically and experimentally by several authors such as Yang et al. [10] and Nogueira and Rodrigues [11].

Previous characterisation has been carried out by Boussafel [12], using the design charts published in both codes: CP110-2:1972 [13] and BS: 8110-3:1985 [14]. The essential parameters taken into account are: $f_{yd} = 410$ MPa for the characteristic yield strength of reinforcement and three values for the characteristic cube strength of the concrete ($f_{cu}$) namely 25, 30 and 40 MPa and a neutral axis depth fixed at $x = 0.5d$, in order to exploit to the maximum the concrete in compression. Characterisations of the equivalent sections in reinforced concrete and reinforced sand concrete have also been established using the material characteristics adopted by Eurocodes 2 and 3 [15, 16].

A characterisation of equivalent reinforced concrete rectangular sections using the characteristics adopted by two codes: CP110-2:1972 and BAEL Rules [17] was established by Boutlitht [18], this characterisation was associated with tests on the pure bending of three IPE beams (IPE 200, IPE 220 and IPE 240). The phenomenon of elastic instability (buckling) of the tested beams was strongly observed. In the majority of the test results, the experimental collapse load was higher than the theoretical collapse load. The rise between experimental and theoretical load varied between 17 % and 39 % for the series of profiles tested [19].

In general, analysis methods often consider reinforced concrete or steel structural elements as linear elastic elements. This assumption, acceptable for the serviceability limit state, is not valid for the ultimate limit state, which is generally characterised by significant cracking and plastification of certain parts of the structure. Consequently, considerable redistribution of forces in the structure and stresses in some elements are probable, which may have large influences on the overall behaviour of the structure in the ultimate limit state. Taking into account the plastic behaviour of the structure seems, therefore, essential to adequately describe and characterize the ultimate limit state of this structure [20, 21].

Two structural elements with the same plastic moment ($M_p$) subjected to the same loading and with the same support conditions have the same theoretical failure load, in other words they have the same bending plastic capacity [22]. The present paper presents an approach to determine simply and doubly reinforced rectangular concrete sections equivalent to the range of IPE sections using the material characteristics adopted by Eurocode 2 [23, 24].

Due to the fact that IPE profiles are produced in the factory and their geometric dimensions and the mechanical characteristics of the metal used are known, they are classified as plastic sections. The plastic modulus of these profiles and thus the plastic moments are directly given in the literature and tabulated. Whereas, reinforced concrete sections having the same plastic moment as a given profile may be multiple. The number of parameters is important (width, height, covering, characteristic strength of the concrete, characteristic strength of the steel, etc.). To overcome these difficulties it is, therefore, absolutely to solve the problem graphically by realizing a catalogue of three-axis design charts linking the reduced moment ($M_d/bd^2$), the ratio of tensioned reinforcements ($\rho_s$) and the ratio of compressed reinforcements ($\rho_c$), using the material characteristics adopted by Eurocode 2, is essential.

In this parametric study and in order to cover all the parameters that can influence the dimensions and percentages of reinforcements of the rectangular section equivalent to a given profile, three practical classes of concrete were selected, namely C25, C30 and C40 having respectively values for the characteristic strength of concrete ($f_{ck}$) 25, 30 and 40 MPa. Also, two practical values for the characteristic yield strength of the reinforcement ($f_{yd}$) have also been taken, namely 400 and 500 MPa. The results of the present characterisation using Eurocode 2 characteristics are compared to those using CP110.

The main objective is to propose the best possible equivalent rectangular section, singly or doubly reinforced with:
- ($\epsilon_{cu} = 3.5 \% \; \epsilon_{st}$ and $\epsilon_{cu} > \epsilon_f$) thus exploiting to the maximum the concrete and reinforcement steel used [25, 26];
- They should also be produced economically (minimisation of the concrete cross-section and reinforcement) [27, 28];
The present analysis was carried out in order to achieve the following objectives:

- Develop a catalogue of design charts similar to the one previously developed by British codes (CP110-2:1972 and BS: 8110-3:1985) [13, 14];
- Carry out a comparative study between the use of the two codes (CP110 and Eurocode 2) in order to formulate an opinion on the change of code;
- Set up an expert mini-system to find the best possible equivalent section in singly or doubly reinforced concrete from the point of view of economic and plastic performance.

The present paper is divided into seven sections. The introduction presents the problem that motivated this research, an overview of previous work as well as a summary of the main objectives are given. The second section presents the basic equations which are absolutely necessary for the graphical realisation of the design charts catalogue. The plastic moments for all IPE profiles are tabulated in section 3. The fourth section describes the procedures used to determine singly and doubly reinforced concrete sections equivalent to the various IPE profiles. The fifth section is devoted to the tabular and graphical presentation of the results. The effect of the different influencing parameters on the dimensioning of the equivalent sections is highlighted in section six. A comparative study between the use of two codes is established in order to highlight the impact of the change of codes. The last section is devoted to the general conclusions of the study. The flowchart of the research and characterisation methodology is presented in Figure 1.

Figure 1. Flowchart of the research and characterization methodology
2. Design and Graphic Development of a Catalogue of Design Charts

2.1. Designing Equations

The design charts, by using the characteristics of the materials adopted by Eurocode 2, are obtained by plotting the reduced moment \( (M_u/bd^2) \), the tensioned reinforcement ratio \( (\rho_t = 100A_t/bd) \) and the compressed reinforcement ratio \( (\rho_c = 100A_c/bd) \) for singly and doubly reinforced concrete rectangular beams and for different values of characteristic steel strength \( (f_yk) \), concrete cylinder strength \( (f_ck) \) and parameter \( d'/d \). This catalogue of design charts is similar to the one published in the British codes (CP110-2:1972 and BS: 8110-3:1985) and these design charts cannot be used to obtain the complete detailed design of any member but they may be used as an aid when analysing the cross section of a member at the ultimate limit state. The charts have been based on the assumptions laid down in BS EN 1992-1-1:2004+A1:2014 [20], use being made of the simplified rectangular stress block based on the Whitney principle which was previously been adopted by BAEL rules [17].

The plotting of this design charts is based on the digitization of the equations obtained by taking the balance of moments about the neutral axis of the section:

\[
M_u = 0.567 f_yk 0.8x (x - 0.8x/2) + f_{sc} A_{sc}(x - d') + f_{st} A_{st}(d - x) \tag{1}
\]

\[
f_{st} A_{st} = 0.454 f_yk b x + f_{sc} A_{sc} \tag{2}
\]

The Equations 1 and 2 can be written as:

\[
\frac{M_u}{bd^2} = 0.454 f_yk \frac{x^2}{d}(1 - 0.40) + f_{sc} \frac{A_{sc}}{bd} \left( \frac{x - d}{d} \right) + f_{st} \frac{A_{st}}{bd} \left( 1 - \frac{x}{d} \right) \tag{1a}
\]

\[
f_{st} \frac{A_{st}}{bd} = 0.454 f_yk \frac{x}{d} + f_{sc} \frac{A_{sc}}{bd} \tag{2a}
\]

For specified ratios of \( A_{sc}/bd, x/d \) and \( d'/d \), the two non-dimensional Equations 1a and 2a can be solved to give values for \( A_{st}/bd \) and \( M_u/bd^2 \) so that a set of design charts such as the one shown in Figure 3, can be plotted. Before the equations can be solved, the steel stresses \( (f_y) \) and \( (f_c) \) must be calculated for each value of \( x/d \). This is achieved by first determining the appropriate strains from the strain diagram (or by applying Equations 3 and 4) and then by evaluating the stresses from the stress – strain curve of Figure 2.

\[
\varepsilon_{st} = \varepsilon_{cc} \left( \frac{d - x}{x} \right) \tag{3}
\]

\[
\varepsilon_{sc} = \varepsilon_{cc} \left( \frac{x - d'}{x} \right) \tag{4}
\]

2.2. Presentation of a Model from the Catalogue of Developed Design Charts

Only one design chart model of the developed catalogue is presented in this article. This catalogue developed for this study contains a series of charts for any value of the parameters \( (f_yk) (25, 30, 40 \text{ MPa}) \) and \( (f_yk) (400, 500 \text{ MPa}) \) and four values for the parameter \( d'/d \) (0.05, 0.10, 0.15, 0.20). The Figure 3 is an example of a design chart for \( (f_yk = 500 \text{ MPa}, f_y = 30 \text{ MPa} \) and \( d'/d = 0.10 \). It is imperative to develop a catalogue of design charts, because the characterisation in an analytical way is almost impossible.

In addition, this catalogue will have two possible uses. The first is when the section is completely defined (geometrically and mechanically), the determination of the ultimate or plastic moment is very easy. The second is when the ultimate moment is known and it will be necessary to reinforce the section optimally.

---

**Figure 2. Design stress – strain curve for steel reinforcement** [29]

---

**Figure 3. Example of a design chart for**

\( (f_yk = 500 \text{ MPa}, f_y = 30 \text{ MPa}, d'/d = 0.10) \).
Figure 3. Typical design chart of the developed catalogue for \( f_{yk} = 500 \text{ MPa}, f_{ck} = 30 \text{ MPa} \) and \( d'/d = 0.10 \)

3. Plastic Moments of all IPE Profile

For the calculation of the internal capacity namely the plastic moment the method of plastic analysis can be used, taking into account the total plastification of the section. Because the geometrical characteristics of the profiles are given, the calculation of the plastic moment is easy, it is just necessary to know the yield strength of the steel used \( (f_y) \). The design plastic moments of the IPE range are given in Table 1. The plastic bending moment of the cross-section is specified in Eurocode 3 [30]. All IPE profiles are classified as bending class 1, therefore their plastic moments are defined by multiplying the plastic bending modulus \( (W_{pl,y}) \) by the steel yield stress \( (f_y) \):

\[
M_{pl,y} = W_{pl,y} \cdot f_y / \gamma_{M0}
\]

(5)

<table>
<thead>
<tr>
<th>Profile</th>
<th>( W_{pl,y} \times 10^3 \text{ mm}^3 )</th>
<th>( M_{pl,y} ) (KNm)</th>
<th>Profile</th>
<th>( W_{pl,y} \times 10^3 \text{ mm}^3 )</th>
<th>( M_{pl,y} ) (KNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IPE 80</td>
<td>23.22</td>
<td>5.46</td>
<td>IPE 270</td>
<td>484.00</td>
<td>113.74</td>
</tr>
<tr>
<td>IPE 100</td>
<td>39.41</td>
<td>9.26</td>
<td>IPE 300</td>
<td>628.40</td>
<td>147.66</td>
</tr>
<tr>
<td>IPE 120</td>
<td>60.73</td>
<td>14.27</td>
<td>IPE 330</td>
<td>804.00</td>
<td>189.02</td>
</tr>
<tr>
<td>IPE 140</td>
<td>88.34</td>
<td>20.76</td>
<td>IPE 360</td>
<td>1019.00</td>
<td>239.50</td>
</tr>
<tr>
<td>IPE 160</td>
<td>123.90</td>
<td>29.11</td>
<td>IPE 400</td>
<td>1307.00</td>
<td>307.18</td>
</tr>
<tr>
<td>IPE 180</td>
<td>166.40</td>
<td>39.11</td>
<td>IPE 450</td>
<td>1702.00</td>
<td>399.92</td>
</tr>
<tr>
<td>IPE 200</td>
<td>220.60</td>
<td>51.85</td>
<td>IPE 500</td>
<td>2194.00</td>
<td>515.62</td>
</tr>
<tr>
<td>IPE 220</td>
<td>285.40</td>
<td>67.07</td>
<td>IPE 550</td>
<td>2787.00</td>
<td>654.95</td>
</tr>
<tr>
<td>IPE 240</td>
<td>366.60</td>
<td>86.16</td>
<td>IPE 600</td>
<td>3512.00</td>
<td>825.41</td>
</tr>
</tbody>
</table>

Where: Steel grade S235: \( (f_y = 235 \text{ N/mm}^2) \) is the yield strength of the profiles and \( (\gamma_{M0} = 1.00) \) is the partial safety coefficient.

4. Determination of Equivalent Reinforced Concrete Sections

4.1. Singly Reinforced Sections

4.1.1. Introduction

The determination of singly reinforced concrete sections equivalent to different IPE must be established by an optimal design. Singly reinforced concrete sections must be under reinforced (Concrete and reinforcement exploited to the maximum \( (\varepsilon_{cc} = \varepsilon_{cu}, \varepsilon_{st} \) and \( \varepsilon_{sc} \) exceeding \( \varepsilon_y \)) with \( x/d = 0.50 \) which corresponds to a plastic strain of the
reinforcement). It is a question of determining the dimensions of the equivalent rectangular section \((b \times h)\) and the tensile reinforcement \((A_{st})\). This is an arduous and complex problem because it consists in determining several unknowns at the same time.

This operation can be carried out by exploiting the catalogue of design charts developed. These design charts must be designed according to the influential parameters, namely the reinforcement covering \(d'/d\), the characteristic compressive strength of concrete \((f_{ck})\), the characteristic yield strength of steels \((f_{yk})\) and the position of the neutral axis \(x/d\), etc. (See Figure 3).

4.1.2. Determination Process

Principles

The process requires knowing and setting the following parameters \((f_{ck}, f_{yk} and d'/d)\) beforehand. It should be noted that each of the design charts in the catalogue developed has been plotted for a combination of these parameters. Thereafter, it is imperative to fix one of the geometric unknowns. For the present study, it was decided to fix the width of the equivalent concrete section \((b)\) in proportion to the width of a given profile \((b_p)\). This proportionality ratio is noted \((\beta = b/b_p)\) and varies from 1.50 to 2.00 with a step of 0.25. This range corresponds to usual and practical rectangular cross-sections. In addition, the useful depth of the equivalent rectangular section \((d)\) must also be fixed. In this study \(d = 0.9 \times h\).

Thus, the process of determining equivalent sections can be started and with an additional requirement for \(x/d\). In fact, for the steel strains to be plastic \((x/d \leq 0.50)\). As for the plastic equivalence, the ultimate moment of the rectangular cross-section \((M_u)\) is taken equal to the plastic moment \((M_p)\) of the profile concerned.

Procedure

Taking the appropriate design chart for a given combination of \((f_{ck}, f_{yk} and d'/d)\). In this design chart, the intersection of the straight line \((x/d = 0.50)\) with the curve \((\rho_{st} = 0)\) gives a point where its horizontal projection gives the value of the reduced moment \((\lambda)\) and its vertical projection gives the value of \((\rho_{st})\).

\[
M_u/b.d^2 = \lambda \tag{6}
\]

From Equation 6 there is:

\[
d = \sqrt{M_u/\lambda. b} = \sqrt{M_u/\lambda. b b_p^2} \tag{7}
\]

So: \(h = d/0.9\);

\[
\rho_{st} = 100 A_{st}/bd \tag{8}
\]

From Equation 8 there is:

\[
A_{st} = \rho_{st} b d/100 \tag{9}
\]

All unknowns \((b, h and A_{st})\) are consequently determined. By varying the values of the geometric ratio \((\beta)\), the characteristic strength of concrete \((f_{ck})\) and the steel yield strength \((f_{yk})\). The result is a series of singly reinforced rectangular concrete sections for the IPE profile range.

Processing an Example

Taking an example, the case of the IPE 270 with \((\beta = 1.50)\), the singly reinforced equivalent section is determined as follows:

- The data for IPE 270: \(M_u = M_p = 113.74 \text{ KN.m}, b_p = 13.5 \text{ cm}\)
- From the appropriate design chart, for example, \((f_{ck} = 30 \text{ MPa}, f_{yk} = 500 \text{ MPa}, d'/d = 0.10)\) and with \((x/d = 0.50)\). The values of the reduced moment \((\lambda)\) and the tensile reinforcement ratio \((\rho_{st})\) can be deduced as follows:

\[
\lambda = M_u/\beta d F = 5.416 \text{ N/mm}^2 \quad \text{and} \quad \rho_{st} = 100 A_{st}/bd = 1.60
\]
- The results for the equivalent singly reinforced concrete section for this example are:

\[
\begin{align*}
\beta &= 1.50, \quad b_p = 20.25 \text{ cm} \\
A_{st} &= 10.43 \text{ cm}^2
\end{align*}
\]

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4.2. Doubly Reinforced Sections

4.2.1. Introduction

Reinforced concrete beams used in practice are usually doubly reinforced sections (i.e. with mounting reinforcements). These doubly reinforced sections must be designed by exploitation to the maximum the concrete and tension and compression steel, the determination of the dimensions \((b \times h)\) of these equivalent sections as well as their reinforcement areas \((A_{st} \text{ and } A_{sc})\), must follow the steps below:

1) The existing data are:
   - The mechanical characteristics of the materials used, such as \((f_{ck}) \text{ and } (f_{yk})\);
   - The ultimate moment \((M_u)\) which is taken equal to plastic moment \((M_p)\) of a given profile;
   - The dimensions of the singly reinforced sections obtained \((b \text{ and } d)\).

2) The determination of the reinforcement areas \((A_{st} \text{ and } A_{sc})\) as well as the dimensions of the doubly reinforced section \((b \times h)\) can be obtained analytically in this part. Using a simplified rectangular stress block based on the Whitney principle which was adopted by Eurocode 2 [20] and previously by BAEL rules [17], with a bilinear diagram for steel reinforcement (see Figure 2).

4.2.2. Determination Process

Principles and Procedure

The design of the doubly reinforced sections is established with a depth of the neutral axis \((x/d=0.50)\), (this limit of 0.50 was taken in order to be able to make a comparison with the characterization established in previous contribution) [12]. This depth of the neutral axis ensures that the strains of the tensioned and compressed reinforcements are in the plastic range, \((\epsilon_u > \epsilon_y, \epsilon_u > \epsilon_y \text{ and } \epsilon_u = 0.0035)\), it is the optimal exploitation of the materials used.

For a singly reinforced concrete rectangular section, the resistant moment is calculated:

\[
M_{r1} = 0.181 f_{ck} bd^2
\]

(10)

With: \(z = d - 0.4 x \) and \(x = \frac{\epsilon_u}{\epsilon_u + \epsilon_y} d\) \hspace{1cm} (11)

First of all, the resistant moment \((M_{r1})\) of the singly reinforced section is calculated. Then comparing the \((M_u = M_p)\) with \((M_{r1})\), when \(M_u \leq M_{r1}\), the concrete section must be re-dimensioned and redesigned, modifying the value of \((d)\) by \((d_{mod})\) where: \((d_{mod} = 0.9 d)\), until \((M_u)\) is greater than \((M_{r2})\), \((M_{r2} = \text{the new resistant moment of the modified section})\), hence the need to use compressed reinforcements.

\[
M_{r2} = 0.181 f_{ck} b d_{mod}^2
\]

(12)

Depending on the internal equilibrium of the moments, the compressed reinforcement area \((A_{sc})\) can be deduced as follows:

\[
A_{sc} = \frac{M_u - 0.181 f_{ck} b d_{mod}^2}{0.87 f_{yk} (d_{mod} - d')}
\]

(13)

Knowing that \(d'd_{mod} = 0.10\), hence:

\[
A_{sc} = \frac{M_u - 0.181 f_{ck} b d_{mod}^2}{0.783 d_{mod} f_{yk}}
\]

(13a)

According to the equilibrium of forces, the tensioned reinforcement area \((A_{st})\) is given by:

\[
A_{st} = \frac{0.227 f_{ck} b d_{mod} + 0.87 f_{yk} A_{sc}}{0.87 f_{yk}}
\]

(14)

Processing an Example

Taking the same example, the case of IPE 270 with \((\beta = 1.50)\), the doubly reinforced equivalent section is determined as follows:

- The data for the singly reinforced section equivalent to IPE 270 are: \(M_u = M_p = 113.74 \text{ KN.m}, b_p = 13.5 \text{ cm}, b = \beta b_p = 20.25 \text{ cm}, d = 32.2 \text{ cm}, f_{ck} = 30 \text{ MPa and } f_{yk} = 500 \text{ MPa}\)
5. Results

5.1. Tabular Presentation of Results

All the different results of this theoretical analysis are presented in different tables, each table containing the dimensions of the singly and doubly reinforced concrete equivalent sections as well as their reinforcement ratios for the different influential parameters ($f_{sk}$, $f_{yk}$ and $\beta$). In this article only two tables are presented. Table 2 represents a model for singly reinforced sections and Table 3 represents a model for doubly reinforced sections.

Table 2. Singly reinforced concrete rectangular equivalent cross-sections for $f_{sk} = 500$ N/mm², $\beta = 1.50, x/d = 0.50$

<table>
<thead>
<tr>
<th>$f_{sk}$ (N/mm²)</th>
<th>$M_p$ (kN.m)</th>
<th>$b$ (cm)</th>
<th>$d$ (cm)</th>
<th>$h$ (cm)</th>
<th>$A_s$ (cm²)</th>
<th>$d_{mod}$ (cm)</th>
<th>$h$ (cm)</th>
<th>$A_{sc}$ (cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>10</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
<td>80</td>
</tr>
</tbody>
</table>

Table 3. Doubly reinforced concrete rectangular equivalent cross-sections for $f_{sk} = 500$ N/mm², $\beta = 1.50, x/d = 0.50$

<table>
<thead>
<tr>
<th>$f_{sk}$ (N/mm²)</th>
<th>$M_p$ (kN.m)</th>
<th>$b$ (cm)</th>
<th>$d$ (cm)</th>
<th>$h$ (cm)</th>
<th>$A_s$ (cm²)</th>
<th>$d_{mod}$ (cm)</th>
<th>$h$ (cm)</th>
<th>$A_{sc}$ (cm²)</th>
</tr>
</thead>
</table>
5.2. Graphical Presentation of Results

5.2.1. Introduction

The cross-sectional dimensions equivalent to the individual IPE profiles obtained from this plastic analysis represent rectangular cross-sections that are usual in practice. The adequate exposure of the different results is the graphical presentation or more precisely the development of a series of curves by scanning the main influencing parameters. Their exploitation makes it easy to determine the most appropriate equivalent sections.

5.2.2. Choice of the Adopted Coordinate System

The choice of the coordinate system obeys to major constraint, that of having all the results on the same graph. The most judicious way is to opt for the logarithmic coordinate system. The best system that has proved successful is that of presenting the results by means of curves with three systems of coordinates.

The entire range of IPE profiles is shown on the x-axis. On the left side of the y-axis, the values $\ln(A_c)$ and $\ln(A_p)$ are shown, where: $(A_c)$ is the area of the equivalent reinforced concrete section, $(A_p)$ is the area of the IPE profile. On the right side of the y-axis, the ratios of reinforcement ($\rho_{st} = 100 \frac{A_{st}}{bd}$ and $\rho_{sc} = 100 \frac{A_{sc}}{bd}$) are shown, hence the right scale takes an independent reading from the left one.

5.2.3. Presentation of the Developed Curves

In this analysis, curves are plotted to represent the obtained sections, either singly or doubly reinforced, equivalent to the different IPE profiles. Three classes of concrete (C25/30, C30/37 and C40/50) were considered having respectively the characteristic strengths of concrete, $f_{ck}$ (25, 30, 40 $N/mm^2$). For each class of concrete there are two characteristic strengths ($f_{yk}$) namely 400 and 500 $N/mm^2$.

A typical selection of curves is presented in the present contribution. These curves representing the singly and doubly reinforced equivalent sections are shown in Figures 4 to 13, where the effect of ($f_{ck}$) and ($f_{yk}$) for a variation of the geometric ratio ($\beta$) are highlighted.

![Figure 4. Singly reinforced concrete equivalent sections for: $f_{ck} = 400$ MPa, $\beta = 1.50$](image-url)
Figure 5. Singly reinforced concrete equivalent sections for: $f_{ys} = 400$ MPa, $\beta = 1.75$

Figure 6. Singly reinforced concrete equivalent sections for: $f_{ys} = 400$ MPa, $\beta = 2.00$

Figure 7. Singly reinforced concrete equivalent sections for: $f_{ys} = 500$ MPa, $\beta = 1.75$
Figure 8. Singly reinforced concrete equivalent sections for: $f_{yk} = 500 \text{ MPa}, \beta = 2.00$

Figure 9. Doubly reinforced concrete equivalent sections for: $f_{yk} = 400 \text{ MPa}, \beta = 1.50$

Figure 10. Doubly reinforced concrete equivalent sections for: $f_{yk} = 400 \text{ MPa}, \beta = 1.75$
Figure 11. Doubly reinforced concrete equivalent sections for: $f_{yk} = 400 \text{ MPa}, \beta = 2.00$

Figure 12. Doubly reinforced concrete equivalent sections for: $f_{yk} = 500 \text{ MPa}, \beta = 1.75$

Figure 13. Doubly reinforced concrete equivalent sections for: $f_{yk} = 500 \text{ MPa}, \beta = 2.00$
6. Effects of the Different Variables

6.1. Effects of the Characteristic Compressive Strength of Concrete ($f_{ck}$) and the Ratio ($\beta$)

6.1.1. Introduction

Figures 4 to 13 clearly illustrate the effect of increasing the characteristic compressive strength of concrete ($f_{ck}$) on the total height ($h$) of the equivalent rectangular section for both types of sections (singly and doubly reinforced). It can also be seen that the curves for the three values of ($f_{ck}$) have the same shape in the same graph.

6.1.2. Singly Reinforced Concrete sections

For the selected values of ($\beta$) and three ranges of ($f_{ck}$): i.e. (from 25 to 30 MPa, from 30 to 40 MPa and from 25 to 40 MPa), the effects of the variation of ($f_{ck}$) are given in Table 4. These values represent the average variation for the whole range of IPE.

- The height ($h$) decreases with the increase of ($f_{ck}$) for a given ($f_{yk}$). On the other hand, the tensioned reinforcement ratio ($\rho_{st}$) increases with the increase of ($f_{ck}$).

- The height ($h$) decreasing from 8.42 % to 21.03 %, whereas the ratio of tensioned reinforcement ($\rho_{st}$) increasing from 22.62 % to 61.53 %.

The maximum gain in concrete is around 20 % for the highest concrete strength (40 MPa), whereas the corresponding added reinforcement needed is around 60 % for the same strength. The results considerate coherent since the plastic moment is very slightly affected by the value of ($f_{ck}$) [29].

Table 4. Effects of the increase of ($f_{ck}$) on the height ($h$) and on the ratio of tensioned reinforcements ($\rho_{st}$) for singly reinforced sections

<table>
<thead>
<tr>
<th>$f_{ck}$ (MPa)</th>
<th>$\beta$</th>
<th>25 to 30</th>
<th>30 to 40</th>
<th>25 to 40</th>
<th>25 to 30</th>
<th>30 to 40</th>
<th>25 to 40</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Decrease of height ($h$) in %</td>
<td>Increase of ($\rho_{st}$) in %</td>
<td>Variation of $f_{ck}$ (MPa)</td>
<td>Variation of $f_{ck}$ (MPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>1.50</td>
<td>8.51 %</td>
<td>13.68 %</td>
<td>21.03 %</td>
<td>22.71 %</td>
<td>30.06 %</td>
<td>59.60 %</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>8.53 %</td>
<td>13.65 %</td>
<td>21.02 %</td>
<td>22.78 %</td>
<td>30.03 %</td>
<td>59.65 %</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>8.52 %</td>
<td>13.65 %</td>
<td>21.01 %</td>
<td>22.62 %</td>
<td>30.06 %</td>
<td>59.47 %</td>
</tr>
<tr>
<td>500</td>
<td>1.50</td>
<td>8.42 %</td>
<td>13.54 %</td>
<td>20.82 %</td>
<td>23.05 %</td>
<td>31.27 %</td>
<td>61.53 %</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>8.50 %</td>
<td>13.58 %</td>
<td>20.92 %</td>
<td>23.10 %</td>
<td>31.20 %</td>
<td>61.51 %</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>8.44 %</td>
<td>13.56 %</td>
<td>20.86 %</td>
<td>22.95 %</td>
<td>31.30 %</td>
<td>61.43 %</td>
</tr>
</tbody>
</table>

The effects of the increase of ($\beta$) (the $\beta$ values varying from 1.50 to 2.00) on the total height ($h$) are given in Table 5:

- The total height ($h$) decreases with increasing ($\beta$) for a given set of ($f_{ck}$) and ($f_{yk}$), this decrease has been found constant and is around 13.40 %.

Table 5. Effects of the increase of ($\beta$) on the total height ($h$)

<table>
<thead>
<tr>
<th>Section type</th>
<th>Decrease of height ($h$) in %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{ck}$ (MPa)</td>
</tr>
<tr>
<td>Singly reinforced sections</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>40</td>
</tr>
</tbody>
</table>

The values given in Table 5 represent the average of decrease in height for the three selected values of ($\beta$) (1.50, 1.75 and 2.00) and this for the whole range of IPE.

6.1.3. Doubly Reinforced Concrete Sections

As for doubly reinforced sections, also for the selected values of ($\beta$) and three ranges of ($f_{ck}$): i.e. (from 25 to 30 MPa, from 30 to 40 MPa and from 25 to 40 MPa), the effects of the variation of ($f_{ck}$) are given in Table 6. These values represent the average variation for the whole range of IPE.

- The height ($h$) also decreases with the increase of ($f_{ck}$) for a given ($f_{yk}$). On the other hand, the ratios of reinforcement ($\rho_{st}$ and $\rho_{st}$) increase;
• The height \( (h) \) decreasing from 8.37% to 21.07%, whereas the ratio of tensioned reinforcement \( (\rho_f) \) increasing from 19.10% to 60.37% and the ratio of compressed reinforcement \( (\rho_c) \) increasing from 15.17% to 62.86;

Also for the doubly reinforced sections, the maximum gain in concrete is around 21% for the highest concrete strength (40 MPa), whereas the corresponding added reinforcement needed is around 61% for the same strength.

Table 6. Effects of the increase of \( (f_{sk}) \) on the height \( (h) \) and on the ratios of reinforcements \( (\rho_f \text{ and } \rho_c) \) for doubly reinforced sections

<table>
<thead>
<tr>
<th>( f_{sk} ) (MPa)</th>
<th>( \beta )</th>
<th>Decrease of height ( (h) ) in %</th>
<th>Increase of ( (\rho_f) ) in %</th>
<th>Increase of ( (\rho_c) ) in %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25 to 30</td>
<td>30 to 40</td>
<td>25 to 40</td>
<td>25 to 30</td>
</tr>
<tr>
<td>1.50</td>
<td>8.46 %</td>
<td>13.75 %</td>
<td>21.05 %</td>
<td>19.43 %</td>
</tr>
<tr>
<td>1.75</td>
<td>8.53 %</td>
<td>13.71 %</td>
<td>21.07 %</td>
<td>19.40 %</td>
</tr>
<tr>
<td>2.00</td>
<td>8.43 %</td>
<td>13.70 %</td>
<td>20.97 %</td>
<td>19.26 %</td>
</tr>
<tr>
<td>1.50</td>
<td>8.40 %</td>
<td>13.57 %</td>
<td>20.83 %</td>
<td>19.24 %</td>
</tr>
<tr>
<td>1.75</td>
<td>8.43 %</td>
<td>13.53 %</td>
<td>20.82 %</td>
<td>19.20 %</td>
</tr>
<tr>
<td>2.00</td>
<td>8.37 %</td>
<td>13.59 %</td>
<td>20.82 %</td>
<td>19.10 %</td>
</tr>
</tbody>
</table>

For the doubly reinforced sections, the effects of varying \( (\beta) \) (the \( \beta \) values varying from 1.50 to 2.00) on the total height \( (h) \) are given in Table 7.

• Similarly to the singly reinforced sections, the total height \( (h) \) decreases with increasing \( (\beta) \) for a given set of \( (f_{sk}) \) and \( (f_{vk}) \), this decrease has been found constant for all tested sections and is around 13.39 %.

Table 7. Effects of the increase of \( (\beta) \) on the total height \( (h) \)

<table>
<thead>
<tr>
<th>Section type</th>
<th>Decrease of height ( (h) ) in %</th>
<th>( f_{sk} ) (MPa)</th>
<th>( f_{sk} = 400 ) MPa</th>
<th>( f_{sk} = 500 ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doubly reinforced sections</td>
<td></td>
<td>25</td>
<td>13.42 %</td>
<td>13.42 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>13.39 %</td>
<td>13.39 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40</td>
<td>13.34 %</td>
<td>13.40 %</td>
</tr>
</tbody>
</table>

6.2. Effects of the Variation of the Characteristic Yield Strength of Reinforcement \( (f_{sk}) \)

6.2.1. Singly Reinforced Concrete Sections

For a given \( (f_{sk}) \) and for the same value of \( (\beta) \), the effects of the increase of the characteristic yield strength of reinforcement \( (f_{sk}) \) on the total height \( (h) \) of the equivalent sections seems to be minor and negligible (see Table 8).

Table 8. Effects of the increase of \( (f_{sk}) \) on the height \( (h) \) for singly reinforced sections

<table>
<thead>
<tr>
<th>Section type</th>
<th>Variation of height ( (h) ) in %</th>
<th>( f_{sk} ) (MPa)</th>
<th>( f_{sk} = 400/500 ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Singly Reinforced sections</td>
<td></td>
<td>25</td>
<td>- 0.08 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>0.00 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40</td>
<td>- 0.14 %</td>
</tr>
</tbody>
</table>

• The effects of the variation of the characteristic yield strength of reinforcement \( (f_{sk}) \) on the tensioned reinforced ratio \( (\rho_f) \) are presented in Table 9. The tensioned reinforced ratio \( (\rho_f) \) decreases with the increase of the characteristic yield strength of the reinforcement \( (f_{sk}) \). For a given value of \( (\beta) \) and for different values of \( (f_{sk}) \) (25 MPa, 30 MPa and 40 MPa) the decrease in reinforcement needed has been found constant and is around 19.82 %.

Table 9. Effects of the variation of \( (f_{sk}) \) on the tensioned reinforcement ratio \( (\rho_f) \) for singly reinforced sections

<table>
<thead>
<tr>
<th>( f_{sk} ) (MPa)</th>
<th>Values of ( (\rho_f) ) for different ( (f_{sk}) )</th>
<th>Decrease of ( (\rho_f) ) in %</th>
<th>( f_{sk} = 400/500 ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>1.63</td>
<td>1.30</td>
<td>20.25 %</td>
</tr>
<tr>
<td>30</td>
<td>2.00</td>
<td>1.60</td>
<td>20.00 %</td>
</tr>
<tr>
<td>40</td>
<td>2.60</td>
<td>2.10</td>
<td>19.23 %</td>
</tr>
</tbody>
</table>
6.2.2. Doubly Reinforced Concrete Sections

For the doubly reinforced sections, for a given ($f_{ck}$) and for the same value of ($\beta$), the effects of the increase of the characteristic yield strength of reinforcement ($f_{yk}$) on the total height ($h$) seems to be also minor and negligible (see Table 10):

Table 10. Effects of the increase of ($f_{yk}$) on the height ($h$) for doubly reinforced sections

<table>
<thead>
<tr>
<th>Section type</th>
<th>Variation of height ($h$) in %</th>
<th>$f_{ck}$ (MPa)</th>
<th>$f_{yk}$ 400/500 (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doubly reinforced sections</td>
<td>+ 0.08 %</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.00 %</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- 0.16 %</td>
<td>40</td>
<td></td>
</tr>
</tbody>
</table>

- The effects of the variation of the characteristic yield strength of reinforcement ($f_{yk}$) on the ratios of reinforcement ($\rho_{st}$ and $\rho_{sc}$) are presented in Table 11. The ratios of reinforcement ($\rho_{st}$ and $\rho_{sc}$) decrease with the increase of the characteristic yield strength of the reinforcement ($f_{yk}$). For a given value of ($f_{ck}$), this decrease appears to be also constant and is around 20 % for ($\rho_{st}$) and around 19.63 % for ($\rho_{sc}$):

Table 11. Effects of the increase of ($f_{yk}$) on the ratios of reinforcement ($\rho_{st}$, and $\rho_{sc}$) for doubly reinforced sections

<table>
<thead>
<tr>
<th>$f_{ck}$ (MPa)</th>
<th>Values of ($\rho_{st}$) for different ($f_{yk}$)</th>
<th>Decrease of ($\rho_{st}$) in %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{yk}$ = 400 MPa</td>
<td>$f_{yk}$ = 500 MPa</td>
</tr>
<tr>
<td>25</td>
<td>1.97</td>
<td>1.58</td>
</tr>
<tr>
<td>30</td>
<td>2.36</td>
<td>1.89</td>
</tr>
<tr>
<td>40</td>
<td>3.16</td>
<td>2.52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$f_{ck}$ (MPa)</th>
<th>Values of ($\rho_{sc}$) for the different ($f_{yk}$)</th>
<th>Decrease of ($\rho_{sc}$) in %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{yk}$ = 400 MPa</td>
<td>$f_{yk}$ = 500 MPa</td>
</tr>
<tr>
<td>25</td>
<td>0.34</td>
<td>0.28</td>
</tr>
<tr>
<td>30</td>
<td>0.40</td>
<td>0.32</td>
</tr>
<tr>
<td>40</td>
<td>0.56</td>
<td>0.44</td>
</tr>
</tbody>
</table>

6.3. Comparative Study between the use of Two Codes

6.3.1. Introduction

A similar characterisation was established in Boussafel (2003) studies [12] in which the design charts published in Part 2 of CP110 were used. This study was carried out for a characteristic yield strength of the reinforcement ($f_{yk}$ = 410 MPa) and for three values of the characteristic cubic strength of the concrete ($f_{cu}$) namely 25, 30 and 40 MPa with a neutral axis depth fixed at ($x = 0.50 \, d$). However, the present study is based on the characteristics of the materials adopted by Eurocode 2 and extended for a variation of ($f_{yk}$) between 400 and 500 MPa for the characteristic yield strength of the reinforcement and an ($f_{ck}$) taking the following values: 25, 30 and 40 MPa for the characteristic cylinder compressive strength of the concrete. The design charts used in this analysis must be designed and developed (a model of these design charts is shown in Figure 3).

6.3.2. Effect of Code Change on the Height of Equivalent Sections

Singly Reinforced Sections

In order to compare the sections obtained from the characterization carried out using two different codes. New design charts have been developed for a characteristic yield strength of reinforcement ($f_{yk}$ = 410 MPa) and for characteristic cylinder strength ($f_{ck}$) taking three values of 20, 25 and 32 MPa (EC 2) corresponding respectively to characteristic cubic strengths ($f_{cu}$) 25, 30 and 40 MPa (CP110), because in the strength classes defined in the Eurocodes (C20/25 to C50/60) the ratios $f_{ck}/f_{cu}$ range from 0.78 to 0.83. An example of this comparison is shown in Figure 14, for ($f_{yk}$=400 MPa and $\beta$ =2.00), where the effect of the code change on the total height ($h$) of the equivalent sections is clearly shown.
According to the results, for a \( f_{ck} \) adopted by Eurocode 2 corresponding to a \( f_{cu} \) adopted by CP110 and for the same \( f_{yk} \), the heights of the equivalent sections obtained using CP110 are greater than those obtained using Eurocode 2. The effect on the height of the sections \( h \) and on the tension reinforcement ratio \( \rho_{st} \) is summarised in Table 12.

### Table 12. Effect of code change on height \( h \) and on the tensioned reinforcement ratio \( \rho_{st} \) for singly reinforced sections

<table>
<thead>
<tr>
<th>( f_{yk} (\text{MPa}) )</th>
<th>( \frac{f_{k}}{f_{o}} ) 20/25 (MPa)</th>
<th>( \frac{f_{k}}{f_{o}} ) 25/30 (MPa)</th>
<th>( \frac{f_{k}}{f_{o}} ) 32/40 (MPa)</th>
<th>Variation of the height ( h ) in %</th>
<th>Variation of ( \rho_{st} ) in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>410</td>
<td>+ 2.00 %</td>
<td>+ 1.20 %</td>
<td>+ 1.90 %</td>
<td>+ 5.93 %</td>
<td>- 3.77 %</td>
</tr>
</tbody>
</table>

#### Doubly Reinforced Sections

As for the doubly reinforced sections, the total height \( h \) of the equivalent sections obtained using the CP110 code compared with those obtained using Eurocode 2 is increasing by an average ratio of 2%, (see an example in Figure 15). On the other hand, the ratio of tensioned reinforcement \( (\rho_{st}) \) decreases by about 5.30 %, while the decrease is about 23.80 % for the ratio of compressed reinforcement \( (\rho_{sc}) \).
7. Conclusion

7.1. Importance of Developing the Catalogue of Design Charts

This study highlighted the importance of the design charts catalogue. The development of these design charts in three-axis graphs linking the reduced moment ($M_u/bd^2$) and the ratio of tensioned reinforcement ($\rho_{st}$) and the ratio of compressed reinforcement ($\rho_{sc}$) using the material characteristics adopted by Eurocode 2 was necessary and indispensable for the following reasons:

- Facilitation of the characterization operation (without this catalogue, the operation would have been impossible to be carried out analytically);
- Facilitation of the comparison between the characterisation obtained by two codes: CP110 and Eurocode 2;
- This catalogue can be used in the design of bent reinforced concrete beams that have rectangular cross-sections in the ultimate limit state (dimensioning and reinforcing);
- It can also be used to quickly check the quality of a bent singly reinforced concrete section (is it under- or over-reinforced);
- Finally, this catalogue allows to immediately determine the flexional capacity of a rectangular section if all the parameters ($f_{ck}, f_{yk}$, and $d'/d$) of this section are known.

7.2. Characterization

From this characterisation, the study revealed the following points:

- All singly or doubly reinforced concrete sections equivalent to the various IPE profiles are in accordance with the sections used in practice;
- The three-axis graphs developed in this study set up a simple mini-system allowing the determination of singly and doubly reinforced concrete sections equivalent to the different IPE profiles which could facilitate decision making;
- Due to the large number of influential geometrical ($d'/d, \beta = b/b_p$) and mechanical ($f_{ck}, f_{yk}$) parameters, the characterisation allowed to obtain a multitude of reinforced concrete sections equivalent to a given profile and it is up to the user to opt for a practical choice;
- All that has been taken into account in the present study is the determination of practical rectangular reinforced concrete equivalent sections to different IPE profiles. The performance and the economy question have been left for future study.

7.3. Importance of Using a Given Code

The comparative study between the use of material characteristics adopted by two codes (CP110 and EC2) has shown that there are more or less important impacts on the equivalent sections obtained. This study revealed the following points:

- The variations are minor for the height ($h$), the percentage varies from (+1.2% to +2.0%) for singly reinforced sections and about (+2.0%) for doubly reinforced sections;
- The variations are more or less important for tensioned reinforcement ratio ($\rho_{st}$), the ratio varying from (-5.93% to +2.0%) for singly reinforced sections and about (-5.3%) for doubly reinforced sections. On the other hand, for the compressed reinforcement ratio ($\rho_{sc}$), the percentage is approximately (-23.8%).

8. Nomenclature

- $A_c$: Area of the equivalent reinforced concrete section
- $A_p$: Area of the IPE profile
- $A_{sc}$: Cross-sectional area of compression reinforcement
- $A_{st}$: Cross-sectional area of tension reinforcement
- $b$: Width of the reinforced concrete rectangular section
- $b_p$: Width of a steel profile
- $d$: Effective depth of tension reinforcement
- $d_{mod}$: Modified effective depth of tension reinforcement ($d_{mod} = 0.9 \, d$)
- $d'$: Depth to compression reinforcement
Characteristic cylinder strength of concrete \[ f_{ck} = (0.78 \div 0.83) f_{cu} \]

Characteristic cube strength of concrete \[ f_{cu} \]

Compressive steel stress \[ f_{sc} \]

Tensile steel stress \[ f_{st} \]

Steel yield stress of a profile \[ f_{y} \]

Characteristic yield strength of reinforcement \[ f_{yk} \]

Overall depth of reinforced concrete rectangular section in plane of bending \[ h \]

Plastic moment of a steel profile \[ M_{pl,y} \]

Resistant moment of a reinforced concrete rectangular section \[ M_{r1} \]

Ultimate moment of resistance or plastic moment of a reinforced concrete rectangular section \[ M_{u} \]

Plastic modulus of a steel profile along the axis of strong inertia \[ W_{pl,y} \]

Neutral axis depth \[ x \]

Equivalent section width to IPE profile width ratio \[ \beta = \frac{b}{b_p} \]

The partial safety coefficient \( \gamma_{M0} = 1.00 \)

Compressive concrete strain \[ \varepsilon_{cc} \]

Boundary compressive concrete strain \[ \varepsilon_{cc} \]

Compressive steel strain \[ \varepsilon_{sc} \]

Tensile steel strain \[ \varepsilon_{st} \]

Yield steel strain \[ \varepsilon_{y} \]

Reduced moment \( \lambda = \frac{M_u}{bd^2} \)

Reinforcement ratio for compression reinforcement \[ \rho_{sc} \]

Reinforcement ratio for tension reinforcement \[ \rho_{st} \]

9. Declarations

9.1. Author Contributions

Conceptualization, S.B. and M.L.S.; writing—original draft preparation, S.B. and M.L.S.; writing—review and editing, S.B. and M.L.S. All authors have read and agreed to the published version of the manuscript.

9.2. Data Availability Statement

The data presented in this study are available in article.

9.3. Funding

The author(s) received no financial support for the research, authorship, and/or publication of this article.

9.4. Conflicts of Interest

The authors declare no conflict of interest.

10. References


Seismic Evaluation of New Steel Infill Panels for Steel Shear Walls

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Received 10 January 2021; Revised 17 March 2021; Accepted 27 March 2021; Published 01 April 2021

Abstract

Corrugated Steel Shear Wall (CSSW) is an efficient shear wall system, which has higher energy dissipation capacity, ductility and stiffness when compared to the Steel Plate Shear Wall (SPSW) with flat infill plate. Despite of these advantages, the ultimate load of CSSW is lower than that of SPSW. Various studies conducted to improve the cyclic behavior of CSSW revealed that increasing corrugation angle might enhance energy dissipation capacity and toughness of CSSWs. However, the ultimate load of CSSW was not improved by increasing the corrugation angle. Thus, the current study proposed new corrugated infill panel schemes to improve the ultimate load of CSSWs. To this end, Finite Element (FE) models were established using ABAQUS/Standard and verified with the experimental results from previous researches. The corrugation angle of the proposed plates was found based on a numerical investigation on seven CSSW FE models with the corrugation angle ranges from 30° to 120°. The FE results revealed that the model with the corrugation angle of 120° achieved highest ultimate load, energy dissipation capacity and toughness amongst the CSSW models. In addition, the ultimate loads, energy dissipation capacities and toughness of the proposed infill plates were up to 11.8%, 53.9% and 8.8% respectively higher than those of CSSW model with the corrugation angle of 120°. Furthermore, the proposed infill plates use up to 13.4% lower amount of steel compared to the corrugated plate with the corrugation angle of 120°.

Keywords: Steel Plate Shear Wall; Corrugated Steel Plate; Corrugation Angle; Cyclic Loading, Finite Element.

1. Introduction

Steel Plate Shear Wall (SPSW) is an economic and highly efficient lateral load resisting system suitable for steel structures in seismic hazard zones, due to its high strength, ductility, stiffness and energy absorption capacity. The steel shear wall consists of boundary frame and an infilled steel plate. The behavior of SPSWs is mostly affected by the early elastic buckling of the infilled plate. Several experimental and numerical studies revealed that welding stiffeners to the infill steel plate could improve the stiffness and strength of SPSWs [1–3]. However, welding stiffeners is a time-consuming procedure and mainly increase the cost of construction. Using corrugated steel plate is another way to improve the buckling resistance of the thin steel plate. To date, several numerical and experimental studies have investigated the behavior of Corrugated Steel Shear Walls (CSSW) with curved and trapezoidal corrugated steel plate under monotonic and cyclic loading. Based on the comparison of the numerical results [4–6], trapezoidal CSSWs depicted slightly higher ultimate load, energy dissipation and toughness compared to the curved CSSWs. Furthermore, the connection between thin steel plate and boundary frame is easier for the trapezoidal corrugated plate.

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http://dx.doi.org/10.28991/cej-2021-03091678

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In Recent decade, numerous experimental and numerical studies have established to evaluate the behavior of the trapezoidal CSSWs. Berman and Bruneau [7] studied the behavior of SPSWs under cyclic loading using three half-scale specimens including a corrugated infill plate and two flat infill plate specimens by means of an experimental research. Shon et al. [8] experimentally studied the behavior of two vertically- and horizontally- trapezoidal CSSW specimens under cyclic loading. The experimental results showed that the corrugation direction did not make much difference in the structural behavior and energy dissipation capacity of the specimens, while it affected the failure mechanism.

Emami et al. [9] performed an experimental study on the cyclic behavior of the trapezoidal CSSWs using two vertically- and horizontally- trapezoidal CSSW specimens and a SPSW specimen with flat infill plate. The obtained results showed that the initial stiffness, energy dissipation capacities and ductility ratios of the CSSW specimens were about 20%, 40% and 52% higher than the SPSW specimen. However, the ultimate strength of SPSW specimen was 17% higher compared to CSSW specimens. In other researches, the impact of corrugation angle (30°, 45°, 60°, and 90°) and infill plate thickness (1.25, 2, 3 and 4 mm) on the behavior of the horizontally- trapezoidal CSSWs have been numerically investigated [10-12]. It was concluded that increasing the corrugation angle improved more or less the energy dissipation and ductility as well as reducing the pinching effect of the hysteresis loops. However, increasing the corrugation angle did not improve the ultimate load of CSSWs unless when using thicker plate. The experimental and numerical investigation on the vertically- trapezoidal CSSWs also showed that increasing corrosion angle could not enhance the ultimate load of the vertically- trapezoidal CSSWs.

Hosseinzadeh et al. [13] experimentally investigated the performance of vertically- trapezoidal CSSW with the corrugation angle of 30°, 45° and 60°. The results reveal that increasing the corrugation angle from 30° to 60° reduced the ultimate load of the specimens. Fadhil et al. [14] performed a numerical investigation on the trapezoidal CSSW with both vertical and horizontal corrugated plate. The corrugation angle of the infill plate ranged from 10° to 90°. The numerical results indicated that despite the hysteresis loops of the models enlarged by increasing the corrugation angle from 10° to 90°, the highest ultimate load belonged to the models with the corrugation angle of 10°.

The above literatures stated that the ultimate load of CSSW is lower than that of the SPSW with flat infill plate. Increasing corrugation angle is not much efficient on the ultimate load of CSSW. Moreover, increasing corrugation angle and thickness of the infill plate requires consuming more steel material, which consequently increases the construction cost. Hence, an efficient infill panel is needed to earn higher ultimate load, energy dissipation and toughness compared to the corrugated plates.

On basis of the aforementioned problem, this study proposed new infill plate configurations based on a combination of flat and trapezoidal corrugated plate to optimize the amount of material consumption and also to achieve a reasonable ultimate load, energy dissipation capacity, toughness and stiffness. For this purpose, numerical models were developed and verified using the experimental results conducted by Emami et al. [9]. Then, parametric study was conducted on vertically- trapezoidal CSSWs to find the most favorable corrugation angle for designing the proposed infilled plate configuration. Finally, the performance of the CSSW with the proposed infilled plate design was evaluated by comparing its numerical results with the numerical results of vertically- trapezoidal CSSWs and the SPSW made of flat infill plate. Figure 1 shows the methodology of the study.

2. Finite Element Modeling

2.1. Model Description

Nonlinear FE analysis was employed to study the behavior of CSSWs under cyclic loading using ABAQUS/Standard software. The specifications and material properties of all the components of the FE model used in this study were adopted from the experimental test specimen (Sample No. 1 and 2) conducted by Emami et al. [9]. The first specimen was made of a boundary frame and a flat infill steel plate, and the second specimen contained a similar boundary frame and a vertically corrugated infill steel plate as detailed in Figure 2 and Figure 3, respectively. The boundary frame included I sections of HE-B140, HE-B200 and HE-B160, which were used for the top beam, bottom beam and columns, respectively. Moreover, the frame components were reinforced by means of stiffener plates. Steel plate which trapezoidal corrugated with a corrugation depth and flat width of 50 mm and 100 mm, respectively, were used as the infill panel with dimensions of 1480×2000 mm and thickness of 1.25 mm shown in Figure 3.b.
Figure 1. Methodology of the study

Figure 2. Detail of FE model with flat infill steel plate (unit: mm)

Figure 3. Detail of FE model (mm) a) the model with trapezoidally corrugated plate; b) Infill panel with a corrugation angle of 30°
2.2. Material Properties

The material properties of all components in this study were adopted from experimental test conducted by Emami et al. [9]. For the steel material in the FE model, the isotropic bilinear stress-strain relationship was assumed with similar properties for both tension and compression. The yield criteria of Von Mises was used to specify the material yield surface, as well as a related flow rule to evaluate the plastic deformation. The material properties of the components were presented in Table 1. The steel Poisson’s ratio was 0.3.

<table>
<thead>
<tr>
<th>Type</th>
<th>Young’s Modulus E (GPa)</th>
<th>Yield stress $f_y$ (MPa)</th>
<th>Ultimate Stress $f_u$ (MPa)</th>
<th>$f_y/f_u$</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate</td>
<td>210</td>
<td>207</td>
<td>290</td>
<td>0.71</td>
<td>41</td>
</tr>
<tr>
<td>Column</td>
<td>210</td>
<td>300</td>
<td>443</td>
<td>0.67</td>
<td>33</td>
</tr>
<tr>
<td>Beam</td>
<td>210</td>
<td>288</td>
<td>456</td>
<td>0.63</td>
<td>37</td>
</tr>
</tbody>
</table>

2.3. Elements Description

To model beams, columns and plates of steel shear wall, a four-node Shell element with reduced integral S4R has been used that is shown in Figure 4. This element has six degrees of freedom in global coordinates in each node and can model large strains and displacements.

The S4R element is an isotropic element, meaning the same shape functions have been used to calculate the spatial displacement field and element geometry [13]. This element uses an integration point in the middle of its surface which reduces the time of structural analysis and increases the results accuracy. In this element, by default, five integral points by thickness are used, which are sufficient to simulate the elastoplastic behavior of the shell structures.

2.4. Surface Interaction

The interaction between the shear wall component surfaces is one of the most significant parameters needed for efficient modeling of the proposed FE model. In the experimental test, the frame components were connected to each other using welding. The infill plate was welded and bolted to the frame to avoid any probable failure in the connection. In fact, proper design of the connections would be very important since the steel infill panel can undergo post-yield stage if no failure occurs in the connections. Any failure in the connection between the steel plate and the frame can lead to loss of the shear wall strength. Since no failure in the connections was recorded in the experimental study [9], tie constraint was utilized for the connection between all the elements of the FE model with presumption that the connection was properly designed.

2.5. Boundary Condition and Loading Program

Fix support assigned to the base beam's bottom flange in the FE models and the translation of nodes at the top beam restricted in global X–direction, as shown in Figure 5. This scenario was conducted in the experimental test to avoid the uplift force and constrain the horizontal and out-of-plane movements. In the global Z–direction, a tabular displacement load was assigned to the exterior column flange at top of the frame, which reflected the horizontal cyclic loading. The cyclic consequence loading was adopted from the experimental test [9] as indicated in Figure 6.
2.6. Meshing Convergence Study

A convergence research was undertaken for the numerical analysis of the FE models to achieve the correct meshing scale for the proposed FE models. The ultimate load capacities of five FE models with specific number of elements were compared to the experimental result [13]. No major difference was found between the load values of the last two iterations, as shown in Figure 7. Since the software requires a fair amount of system running time on a standard PC with higher number of elements, the models with 2475 and 3153 elements were chosen to reflect the FE models with flat and corrugated infill plate, respectively.
3. Validity of FE Modelling

The accuracy of the FE models was verified by comparing the lateral load vs displacement hysteresis results and failure modes of FE analysis and their corresponding experimental results conducted by Emami et al. [13]. The models with flat infill plate and trapezoidal corrugated steel plate were referred by the specimens No. 1 and No. 2 in the experimental test, respectively. Figures 8 and 9 compare the lateral load vs displacement hysteresis results of the FE analysis and the experimental test, which show the hysteresis loops of the FE analysis were well matched with those of the experimental test. Table 2 presents a comparison between the results obtained from FE analysis and the experimental test, which indicates that the FE analysis overestimated the ultimate lateral loads of the specimens No. 1 and 2 by approximately 1% and 3%. Figure 8 displays the deformed shape of the FE model. Severe buckling of the infill plate was observed in both the experimental test and FE results. In addition, the buckling of triangular plates was also observed in both FE result and the experimental test. Figure 10 shows the deformed shapes of the FE models.

Table 2. Comparison of FE analysis and the experimental test results

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Ultimate load (kN)</th>
<th>FE/EX Ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EX</td>
<td>FE</td>
</tr>
<tr>
<td>No 1</td>
<td>580</td>
<td>595</td>
</tr>
<tr>
<td>No 2</td>
<td>500</td>
<td>507</td>
</tr>
</tbody>
</table>

Figure 8. Comparing the lateral load vs. displacement hysteresis results (specimens No. 1)
In this study, the influence of corrugation angle on the hysteretic behavior, ultimate lateral load, energy dissipation capacity, toughness of CSSWs were investigated by means of seven FE models with corrugation angles (θ) of 30°, 45°, 60°, 75°, 90°, 105° and 120°. For all models, the width of the flat parts and depth of the corrugated infill plate were 100 and 50 mm, respectively, as shown in Table 3. Furthermore, the other parameters such as the material properties, the boundary frame specifications and infill plate thickness were similar to the experimental specimens as described in Section 2.2.

**Table 3. Corrugated plate sections**

<table>
<thead>
<tr>
<th>Model</th>
<th>Section</th>
<th>Angle</th>
<th>a (mm)</th>
<th>b (mm)</th>
<th>c (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C30</td>
<td></td>
<td>30</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>C45</td>
<td></td>
<td>45</td>
<td>100</td>
<td>100</td>
<td>70.71</td>
</tr>
<tr>
<td>C60</td>
<td></td>
<td>60</td>
<td>100</td>
<td>100</td>
<td>57.73</td>
</tr>
<tr>
<td>C75</td>
<td></td>
<td>75</td>
<td>100</td>
<td>100</td>
<td>51.76</td>
</tr>
<tr>
<td>C90</td>
<td></td>
<td>90</td>
<td>100</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>C105</td>
<td></td>
<td>105</td>
<td>100</td>
<td>100</td>
<td>51.76</td>
</tr>
<tr>
<td>C120</td>
<td></td>
<td>120</td>
<td>100</td>
<td>100</td>
<td>57.73</td>
</tr>
</tbody>
</table>
In order to identify the model’s properties easily (the type and corrugation angle of the infill plate), the CSSW models were labeled with letter “C” and the number next to that represents the corrugation angle. While, the FE model with flat infill plate labeled with “Fl”.

Figure 11 displays the hysteresis curves of the FE models. As observed in Figure 11, the pinching effect in the hysteresis loops of the models was reduced with an increase of the corrugation angle, which leads to wider and spindle-shaped loops.

![Hysteresis Curves](image)

Table 4 presents the ultimate loads of the FE models. The comparison of the results show that the ultimate load of the models decreased 16.9% as the corrugation angle increased from 30° to 60°, and then improved more or less by increasing the corrugation angle from 60° to 120°. The experimental research on the vertically-trapezoidal CSSWs also showed that the CSSWs gained lower ultimate load as the corrugation angle increased from 30° to 60° [13].

Figure 11. Hysteretic curves of the FE models with different corrugation angles

Table 4 presents the ultimate loads of the FE models. The comparison of the results show that the ultimate load of the models decreased 16.9% as the corrugation angle increased from 30° to 60°, and then improved more or less by increasing the corrugation angle from 60° to 120°. The experimental research on the vertically-trapezoidal CSSWs also showed that the CSSWs gained lower ultimate load as the corrugation angle increased from 30° to 60° [13].
other study on the vertically-trapezoidal CSSWs, Fadhil et al. [14] also found a reduction in the ultimate load of the FE models by increasing the corrugation angle up to 60°, and then the ultimate loads improved with an increase of corrugation angle from 60° to 90°. Furthermore, the hysteresis loops of the FE models got wider and more stable cyclic behavior as the corrugation angle increased, which is also in good agreement with the FE results of the current study. Figure 12 compares the ultimate loads of the CSSW models (Vu-C) with that of the model with flat infill plate (Vu-Fl). It is observed that C30 and C120 achieved Vu-C/Vu-Fl of 0.85, which is highest value amongst the other FE models.

Table 4. The results of the FE models with different corrugation angle

<table>
<thead>
<tr>
<th>Model</th>
<th>Corrugation Angle</th>
<th>Ultimate Load (kN)</th>
<th>Energy Dissipation Capacity (kN.m)</th>
<th>Toughness (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fl</td>
<td>0</td>
<td>595</td>
<td>756.2</td>
<td>36.6</td>
</tr>
<tr>
<td>C30</td>
<td>30</td>
<td>507</td>
<td>1002.4</td>
<td>43.1</td>
</tr>
<tr>
<td>C45</td>
<td>45</td>
<td>452</td>
<td>1038.3</td>
<td>41.4</td>
</tr>
<tr>
<td>C60</td>
<td>60</td>
<td>421</td>
<td>960.6</td>
<td>38.7</td>
</tr>
<tr>
<td>C75</td>
<td>75</td>
<td>469</td>
<td>1084.1</td>
<td>43.3</td>
</tr>
<tr>
<td>C90</td>
<td>90</td>
<td>466</td>
<td>1043.2</td>
<td>41.2</td>
</tr>
<tr>
<td>C105</td>
<td>105</td>
<td>483</td>
<td>1134.6</td>
<td>43.9</td>
</tr>
<tr>
<td>C120</td>
<td>120</td>
<td>505</td>
<td>1152.9</td>
<td>45.6</td>
</tr>
</tbody>
</table>

Figure 12. Comparison of the ultimate loads of the CSSW models with Fl

Table 4 also presents the energy dissipation capacities of the FE models. It is observed that except in the case of 60° and 90°, increasing the corrugation angle from 30° to 120° enhanced the energy dissipation capacities of the FE models up to 15%. Figure 13 shows the ratios of the energy dissipation capacities of the CSSW models to the energy capacity of Fl (DE_C / DE_Fl). From Figure 13, C60 achieved DE_C / DE_Fl of 1.27, which was the lowest value of DE_C / DE_Fl among the other CSSW models. The values of DE_C / DE_Fl revealed 52% improvement in the energy dissipation capacities of the CSSW models compared to the energy capacity of Fl, when the corrugation angle of infill plate was 120°.

The toughness of the FE models is provided in Table 4. The results show that the model’s toughness reduced 10.2% by increase in the corrugation angle from 30° to 60° and increased 17.8% as the corrugation angle increased from 60° to 120°. Figure 14 compares the CSSW model’s toughness (T_C) with that of Fl (T_Fl). The results of T_C/T_Fl indicated that C30 achieved T_C/T_Fl of 1.18%. Whilst, increasing the corrugation angle from 30° to 120° increased the T_C/T_Fl to 1.25.
Based on the comparison of the results, C120 achieved the highest ultimate load, energy dissipation energy capacity and toughness amongst the FE models with corrugated infill plates. Moreover, although the ultimate load of C120 was 15% lower than that of Fl, the energy dissipation capacity and toughness of C120 were 52% and 25% higher compared to those of Fl, respectively. It is noteworthy that the authors numerically investigated the effect of thickness and the height-to-width ratio of the infill plate in an unpublished research. The results showed that the models with the corrugation angle of 120° gained the highest ultimate load, energy dissipation capacity and toughness compared to the corresponding models, regardless the thickness and the height-to-width ratio of the infill plate. The ultimate shear force carried by the steel shear walls is determined through two mechanisms, pure shear and diagonal tension field as presented in Equation 1 [9].

\[ V_w = Lt \left( \tau_{cr} + 1/2 \sigma_{ty} sin2\alpha \right) \]  

Where; \( \tau_{cr}, \sigma_{ty}, \alpha, L \) and \( t \) are the critical shear buckling stress, tension field stress, tension field angle, total length of flat sub panel, and thickness of the infill plate, respectively.

The pure shear in SPSWs with a thick infill plate generates steady plastic cyclic behavior due to widespread shear yielding. Therefore, pinching effects would be reduced in the hysteresis loops [3]. The pure shear mechanism could be improved by increasing corrugation angle due to increase in critical buckling shear stress. It could also cause a
reduction in the tension field mechanism due to increase of the tension field inclination angle, as shown in Figure 15. Therefore, the ultimate load of the models reduced as the corrugation angle increased from 30 to 60 due to the reduction in the tension field mechanism. Meanwhile, the total length of flat subpanels (L) considerably enlarged by increasing the corrugation angle from 60° to 120°, which leads to higher ultimate load.

Figure 15. The tension field inclination angles of the CSSW models
5. Proposed Infill Plate

This study proposed two infill plate configurations to increase the ultimate load of CSSWs as well as optimizing the amount of steel consumption used for the infill plate. The proposed infill panels were a combination of flat and trapezoidal corrugated plates. Figure 16 shows the models with the proposed infill panels, which were labeled with “CS120” and “CM120”, respectively. Berman et al. [15] and Lv et al. [16] reported that the tension field in the infill plate occurs in three zones. Zone one and three were at the side of plate and zone two was at the middle of plate. Each zone has its own tension field inclination angle as shown in Figure 17. In this research, same concepts were utilized to design the proposed infill panels. For CS120, the flat part with the width of one-third of the total infill panel was located at the middle of the panel, while the rest of the panel was corrugated. On the contrary, in case of CM120, one-third of total infill panel width at the middle of the infill panel was corrugated and the rest of the panel kept flat while the flat part with the width of one-third of the total infill panel located at the middle of the panel, while the rest of the panel was corrugated. For CS120, the flat part with the width of one-third of the total infill panel located at the middle of the panel, while the rest of the panel was corrugated. On the contrary, in case of CM120, one-third of total infill panel width at the middle of the infill panel was corrugated and the rest of the panel kept flat. The corrugation angle, depth and flat part of the corrugation were 120°, 50 mm and 100 mm respectively for both models. Furthermore, the other parameters of CS120 and CM120 such as the specification of boundary frame, infill plate thickness and material properties were similar to the experimental test as described in Section 2.1.

Figure 16. Configurations of the models with proposed infill plates (a) CS120 (b) CM120

Figure 17. Three Zones of infill plates

Figure 18 illustrates the hysteretic curves of CS120 and CM120, which indicated that CS120 had wider loops and less pinching effect compared to CM120. However, the ultimate load of CM120 was 7.5% higher than that of CS120 as presented in Table 5.

Table 5. FE results of CS120 and CM120

<table>
<thead>
<tr>
<th>Model</th>
<th>Corrugation Angle</th>
<th>Ultimate Load (kN)</th>
<th>Energy Dissipation Capacity (kN.m)</th>
<th>Toughness (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS120</td>
<td>120</td>
<td>524</td>
<td>1742.3</td>
<td>47.1</td>
</tr>
<tr>
<td>CM120</td>
<td>120</td>
<td>563</td>
<td>1767.6</td>
<td>49.9</td>
</tr>
</tbody>
</table>
Figure 18. The hysteretic curves of CS120 and CM120

Figure 19 compares the ultimate loads of C120, CS120 and CM120 ($V_{u,C}$) with $V_{u,Fl}$. The ultimate loads of CS120 and CM120 were 12% and 5% lower than $V_{u,Fl}$, respectively. In addition, the results show that the ultimate loads of CS120 and CM120 were 3.5% and 11.8% higher than that of C120.

Figure 20 indicates the ratios of the energy dissipation capacities of C120, CS120 and CM120 to the energy capacity of the model with flat infill plate ($D_{EC}/D_{EFl}$). The results show that the energy dissipation capacities of CS120 and CM120 were 1.37% and 1.30 times greater than that of Fl. Beside, CS120 and CM120 achieved considerably higher energy dissipation capacities compared to C120 as their energy dissipation capacities were 53.9% and 51.6% greater than that of C120, respectively.
The toughness of C120, CS120 and CM120 ($T_C$) were compared with $T_F$ in Figure 21. The comparison shows that CS120 and CM120 gained the toughness approximately 29% and 36% higher than that of the model with flat infill plate. Meanwhile, the toughness of C120 was 3.1% and 8.1% lower than those of CS120 and CM120, respectively.

![Figure 20. The ratios of the energy dissipation capacities of C120, CS120 and CM120 to the energy capacity of Fl](image)

![Figure 21. Comparison of the toughness of C120, CS120 and CM120 with the toughness of Fl](image)

The results reveal that CS120 and CM120 performed better than C120 as the ultimate loads, energy dissipation capacities and toughness were higher than those of C120. While, the amount of steel consumption for infill plates of CS120 and CM120 were 24.6% and 28.4% less than that of C120, respectively.

The reason of better performance of the proposed infill plates than flat plate through energy absorption and toughness could be attributed to the shear buckling of the plates ($\tau_{cr}$) as presented in Equation 2 [9].

$$\frac{1}{\tau_{cr}} = \frac{1}{\tau_{cr,L}} + \frac{1}{\tau_{cr,G}}$$

$$\tau_{cr,L} = \frac{k\pi E}{12(1-\nu^2)} \left(\frac{t}{a}\right)^2$$

$$\tau_{cr,G} = 36\beta E\left[\left(\frac{d}{t}\right)^2 + 2\right]/\eta \left(\frac{t}{h}\right)^2/12(1+\nu^2)$$

Where, $E$ and $\nu$ are the elastic modulus, Passion ratio, respectively. $\beta$ and $k$ are factors associated with the boundary condition of the infill plate. $\eta$ is defined as $(a+b)/(a+c)$. $a$, $b$, $c$ and $d$ are the width of flat panel, the width of inclined panel, the projected width of inclined panel and corrugation depth, respectively. $h$ is infill panel height.

Based on Equation 2, the infill of CM120 and CS120 gained higher $\tau_{cr}$ compared to Fl, since the flat panel width ($a$) is shorter than that of Fl. Thus, CM120 and CS120 (similar to CSSWs) can reach to greater displacement than Fl, which could lead to higher energy dissipation capacity and toughness. In addition, due to superior shear strength of the
flat plate compared to the corrugated plate, CS120 and CM120 gained higher ultimate loads than that of C120. It’s obvious that the proportion of flat plate can affect the ultimate load of the models. Therefore, the ultimate load of CM120 is greater than that of CS120, as its infill panel specified larger area of flat plate. However, the corrugated plate exhibited better performance in damping the lateral load. Hence, the energy dissipation capacity of CS120 was higher than that of CM120, because the proportion of the corrugated plate used in the infill panel of CS120 was greater. The damping effect of corrugated plate can be observed in the hysteretic behavior of CS120; in which the hysteresis loops CS120 showed minor pinching effect.

6. Conclusions

This study proposed new infill plate designs to improve the cyclic behavior of CSSWs as well as reducing the amount of steel used for infill panel. In this study, combinations of flat and corrugated plates were suggested based on a numerical investigation on the CSSW models with a variation of corrugation angle ranged from 30 to 120. According to the FE results, following conclusions were drawn:

- Based on the numerical investigation on the CSSW models with different corrugation angles, it was concluded that increasing corrugation angle from 30° to 120° improved the energy dissipation capacity and toughness up to 15% and 24.6%, respectively. In addition, C120 and C30 gained highest ultimate loads among the CSSW models.
- The proposed infill plate designs earned satisfactory results as the ultimate loads, energy dissipation capacities and toughness of CS120 and CM120 were up to 11.8%, 53.9% and 8.8% higher than those of C120, respectively. Meanwhile, the infill panels of CS120 and CM120 used 24.6% and 28.4% less steel material compared to that of C120, respectively.
- And it is concluded that using corrugated plate at the middle of the infill panel is a better option as the ultimate load and toughness of CM120 was greater 7.4% and 5.9% than those of CS120, respectively. While, its energy dissipation capacity was negligibly lower than that of CS120. Furthermore, infill panel of CM120 used 13.4% lower steel material compared to that of CS120.

7. Declarations

7.1. Author Contributions

Conceptualization, Methodology, Investigation Data Collection, Writing, Original Draft, Visualization and Validation, A.J.; Writing - Review & Editing M.A.; Review and Supervision, S.A.O.; Review and Supervision M.Y.M.Y.; All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

7.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

7.4. Acknowledgements

I would like to thank Mohadeseh Rahmani Nia for her direction, sound guidance and kind assistance throughout this research.

7.5. Conflicts of Interest

The authors declare no conflict of interest.

8. References


A Case Study of S-Curve Analysis: Causes, Effects, Tracing and Monitoring Project Extension of Time

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Received 16 June 2020; Revised 27 February 2021; Accepted 05 March 2021; Published 01 April 2021

Abstract

S-Curve analysis in the construction interpreted as managing project with knowledge and traceable in the context of sustainable construction while displays the cumulative costs, labour hours or other quantities plotted against time. In the contract administration, delays in completing a construction project led to the breach of contract but, in contracts itself allow the construction period to be extended where there are delays that are not the contractor's fault. Under those circumstances, a presentation of a case-study regarding the analysis of S-Curve of a life project drew comparative interpretation of project performance towards project delivery schedule has been conducted in private initiative project. This study aims to investigate and examine the factors that cause delays in construction projects from the perspective of S-curve representations. The paper aims to provide in depth light about the existing causes of project delay and describe the key sources of financing problem and identify the consequences of contraventions of contract. Two distinct parts divided which are refers to the methods used to assess the perceptions of clients, consultants, and contractors on the relative importance of causes of delay in a project and referred to the procurement and documentation to analyse the delay. As a result, an Extension of Time (EOT) granted and identically changed the progress towards extension time where better planning demanded for improvement and restoration progress kept on track. This paper presented a practical and comparative S-Curve within extension of time to ensure delivery of project on schedule. In the long run, the identified causes are combined into 16 factors. Finally, the result of this match was brought in order to critically understand and provide a guideline to contractor in preparing EOT application and choose reliable factor based on the specific circumstances of project delay factors thorough review conducted to reveal the nature of EOT application techniques.

Keywords: S-Curve; Causes; Extension of Time; Monitoring Project; Management.

1. Introduction

Presentation of a case-study with reference to the analysis of S-Curve to trace of a real project has been marked as complete with regards to successfully project delivered as per schedule. S-Curve has been used as a procurement performance model for construction frameworks to trace and track status of project for guidance in reporting progress
in contract management. The S-curve is quite suitable to represent the relationship between project duration and complete progress in practical usage of construction management [1].

The necessity is compatible to the needs of project analysis and innovation in this paper will be able to assist contractors to provide the EOT application documentation based on identified delay factors based on the specific circumstances of each project. In fact, contractor initiatively to identify the root cause of the problem and determined all the possible consequences of a fact in relationship between different root causes of a problem for detailed parts. After decades, the use of appropriate procurement method to implement construction projects was brought to the fore after the release of a model [2] and subsequently reported evolved [3]. Construction industry in Sabah, Malaysia are on the right track with positive variance of development driven by rapid economic growth and domestic demand. Unfortunately, there are some triggers that interfered with or reduced the smoothness of development teams, among which is delay in project execution. Various issues of delays raised by contractors.

Notwithstanding, the problems of delays in Sabah’s construction industries is not a major problem or unusual phenomenon. It has been researched; the causes of projects being executed in Sabah East Malaysia allowing a mitigation plan to be prepared [4]. In Sabah itself, the enormous developer was driven by government in construction industry and private sector become a minority in development sector to fulfilled domestic demand. Obviously, the biggest customer of the construction industry in most countries is the government [5]. It was observed that the performance of private sector in the construction industries in terms of time was quite manageable in Sabah. However, in construction sector there are tremendously productivity with skills, but overall seemed to industry not merely a solving in delays in execution, but a kind of sufficient human capital. Additionally, building construction often occupies the bottom of industrial productivity rank reports worldwide [6]. Construction typically involves a deadline for work completion that started from commencement date. In general, contractual agreements will be forced attention to schedules and delays in execution will imply construction, represent additional costs due to late facility occupancy or other factors. A study of construction industry delays carried out in Hong Kong, the execution of project related to timely delivery of projects within budget and to the level of quality standard specified by the client is an index of successful project delivery [7].

The traditional approach to estimating an S-curve is based on a schedule of planned activity times and progress calculation using the percent weight of each activity in the project and the percent complete of each activity at each time point [8]. Thus, this case-study conducted review to the contractor’s technique to overcome their delays. In order to overcome the difficulties of controlling projects, the S-Curves based on financial are applicable to project management applications in this situation. Costs versus Time S-Curve be a baseline in monitoring project progress. The schedule and actual percentage are compared in every month to accelerates the remedial plans. Contractors and developer believed the S-Curve as a productive medium in reporting progress. Consequently, the S-type distribution is believed to be suitable in regression on construction management and social economy [1].

The objective of this case-study is to determine the practical cumulative S-Curve that will best represent the project progress report and to improve the performance after first extension of time agreed by developer and contractor by negotiation. In this study, S-Curve representing the standard distribution of costs over time and a cumulative flow of money over a time period. The optimization process of second phase in construction during extension of time are greatly monitored using the S-Curve and doesn’t change the relationship between activities.

This study is discussed the restoration and improvement from the origin S-Curve to the modified S-Curve to suit the first granted extension of time. From the S-Curve, there are several causes and project delays effect widely shared. A remedial action and time to time procurement that need to be supervised, monitored and acquired by the contractor to protect their current interest to make an application for extension of time as be conclusive evidence of contract administration in future. In order to identify the unusual activities in S-Curve, this paper is organized in the section 1, conflict in the construction issues for determination of causes and effect. The basics of S-Curve are analyzed based on the origin S-Curve before extension of time granted in section 2, modified S-Curve according to new regime of time based on the extension period and the core of a progress are presented in section 3.

In many circumstances, delay project was ended with dispute between involved parties from client, consultants and contractor. There are many arguments was present to prove their representations to the court. Determining the contractual responsibility of delay is the most likely source of dispute in construction projects and many techniques have been used in the courts to demonstrate the criticalities of a delay event on the project schedule [26]. Most compelling evidence, finance-related causes as the most critical causes of delay in any projects. Contractors facing difficulties when it comes to financial problem or late of payment and releases of interim certificates. This factor widely discusses by many researchers. Delay in settlement of claims, contractor’s financial difficulties, delay in payment for extra work/ variations by owner, late payment from contractor to subcontractor or suppliers, variation orders/changes of scope by owner during construction and changes in design by owner were the highly ranked delay causes [27].
Poor consultant performance and inefficient site management discovered as a part of contributor to the causes of project delay. Inefficient will invite major problem in management. Consultants must perform their professionalism in well verse as middle party between client and consultant. Failure in delivering good performance causes improper project implementation. Six of the ten most important causes are in the top ten universal delays in construction projects. Factor analysis revealed six underlying causes: improper planning, poor consultant performance and inefficient site management [28]. To ensure project delivery on time, ability to control the risk of delays is very important. About 60% of project less delay, this happened when the effectiveness and managing the risk carefully. Attention given to project knowledge management, record and review past events, record keeping, information evaluation, data ratings, quantify their importance, utilize expert knowledge in rooting and data analysis [29].

2. Background of Case Study

A case study of S-curve analysis by planning, tracing and monitoring model during project extension of time are carried out in a construction package located in Sabah, Malaysia. A contract has been signed represent third parties between developer, consultant and project has awarded to local main contractor. Project is developed under private initiative scheme with cost USD$ 30 Million in rural area to support positive economic, social and environmental links between urban and rural areas by strengthening state and regional development planning. The proposed development is a major commercial development that are set to change the face of state rural area. The development not only big in terms of size and gross development value but also special in concept and design that makes new township area for daily activities. With type of building known 2 and 3 storey shop lot. The contract period are 26 weeks. Unfortunately, due to unforeseen scenario the project undeliverable and contractor has decided to apply the first extension of time (EOT) and conditional approval are granted. This project is behind 25.22% from schedule.

This approval is granted based on a thorough review of all information available and facts related to project. The developer unanimously agreed to grant preliminary conditional approval to contractor with proven statement by the causes of delay in project. Contractor has managed to prepare probe procurement that become a backbone to the application. Strong recommendation finally made by consultant after careful review. The granting of an extension of time relieves the contractor from penalty for failure to achieve the contract period and free from legal action to enforce against developer rights. The contractor is granted the appropriate extension of time for 6 months from the duly completion date. Contractor has the right to claim an extension of time which limits the time within which certain contractual rights can be enforced. In addition, the contractor has the right to assume any missing information, but all assumption made should be qualified clearly in the schedule narrative that is submitted along with the schedule [8].

The development plan has prepared by consultant accordingly and contractor has the right on their obligation to follow the shown plan. In consequence, the interpretation of S-Curve is derived from this figure. In this study, contractor has issued claim on extension of time for the late delivery from various causes are presented and documented and widely discussed in the next section. The contractor scope of work applies to the development project as stated in formal plan sheets and construction documents and clearly stated, as main contractor their obligation and work are to complete the physical development such as building under building works contract including mechanical and electrical services.

Scope, budget and schedule constraints are core characteristics of the production of projects [9]. At the early stage of construction, contractor moved smoothly and time to time, they are facing problem and occasionally disturbed by few internal discrepancy issues. Contractor wisely manage the arising matter and prepared any written instructions for every event and obviously the contractor show their professional experienced as a good practice. The key findings reveal that in achieving the best practice in the Malaysian Construction Industries, project manager should straighten their capability in term of knowledge, skills and personal characteristics [10]. A well-organized development programme is a critical strategy for construction companies [11].

During the construction period, as a contractor there will be question running in mind whether, will the project be completed as scheduled. Thus, from the experienced, contractor has taken consideration from any angle and perspective to use opportunity to fully. All work requests will be prioritized by contractor. Approaches to controlling performance cost, time and quality are examined, the importance of working to a programme and to a preconceived specification is stressed [12]. Figure 1 shows the research methodology and conducting analysis from various parties involving client, consultant and contractor. Analysis including documentation extractions and interim certificate analysis.
3. Conflicts, Causes and Effect

Understanding the causes and effects of conflict is fundamental to personal and professional management skills in construction. It happened under the pressure in every angle of project and seemed the most normal. Conflict seems to be very synonym with construction projects and giving the impressions of problems includes in increasing project cost, project delays, reduce productivity, loss of profit or damage in business relationships [13]. Like any construction project, the development of this project has been a team effort. Unfortunately, the construction team are beset with disputes in professional way. Generally speaking, most conflict begins when there is a difference. This argument increases the possibility of delay in progress by dispute decided that sometimes taking time to settle at the contractor's risk. Poor site management influencing factors in causing delay arranged in descending order [14].

In this case-study, the mentioned project is faced with some conflicts. This report on project delivery and lessons learned, in undertaking the granted extension of time, contractor take the first step started to identify a problem, investigate and finding possible solutions to problem. Professionally, contractor manage the conflict by not taking into account personal interests or conflicts and they come with conclusive idea to solve the delay. Contractor comes with monitoring plan using financial S-Curve reports. Prior to the implementation of new S-curve based on the granted extension of time, there are three conceptual ideas decided in identifications problem and solution firstly, identify the root causes of delay. Secondly, restructuring of manpower and thirdly, monitoring by using financial S-Curve as crucial reporting medium to developer and as indicator to the warning strength of team and allow the progress of a project to be tracked visually over time. In projects the S-curve is driven by the multiple interconnected activities that occur in the middle of a project [15]. In what follows, this paper only discussed the first and third conceptual ideas that highlighted by contractor. Where, causes and S-curve are eligible to be discussed with generated a S-curve in construction studies to fit project management data. Table 1 shows the factors causing delay in this project. There are 16 major points that contributed to the delay root causes from particular parties.

Contractually, contractor should be handing over this whole project by middle of 2015 unfortunately, as per schedule, this project determined behind the schedule at the stage of second year at December 2014 after 21 months’ progress of works. After 21 months, this project actual percentage identified behind 5.69 from total 80.47% schedule
to actual only 74.78% only. This phenomenon has been decided by client, consultant and contractor as unusual progress where clearly stated in contract and breach the terms. This regards to the Sale and Purchase Agreement between clients to customer where, there will be a penalty for late delivery. According to clause, developer guarantee that the buyer will receive the compensation for late delivery. Nonetheless, in contract itself has stated that, in any condition that proven, contractor may apply the Extension of Time (EOT) for their non-cause problem if the project conditionally determined as behind schedule after 70% of work progress. Contractor decided to follow up the progress by putting forward the causes. The weightage of delay percentage presented in Table 1 based on the total number of days taken by each issue to be solved.

EOT is one of the provision clauses in the standard contract form. The purpose is to preserve an employer’s right in liquidated damages. In the circumstances that the delays are caused by inevitable reason, EOT allows contractor to set an agreed completion date [16]. Slow decision making by particular parties with 1.12% mean index of delay cause. The most preferred factor as causing delay in this project with mean index 3.74% are bad weather with total number of rainy days is 443 days during the construction period. Contractor has been advised to get the rainfall official data associated the locality of project as supported documents in applying EOT. This is closely followed by the shortage of skills worker with mean index 2.76% as part of the delay contributor and weightage of S-Curve derivation.

At the beginning of handing over the site construction, the main physical contractor has recorded in delay for almost three months started from January to March 2013 for late delivery from earth work contractor with 2.22% factor. The developer aware and put into record the delay causes as a compulsory option in consideration of approval EOT. Discrepancies of drawing and specifications happened between architecture and structural drawing that involved both parties architect and consultant. Document review and checking should always be under the responsible of contractor and should reporting any discrepancies to the architect or engineer be mandatory. In this case-study, the mean weightage delay causes by these issues contributed 2.04% and seen has lots of effect on project. The problem that arises is that discrepancy between drawings and specifications often occur [17]. Variation order weightage 1.91 and is not a major cause but it always viewed in changing, disturbing the main task significantly and expanding time to the additional works that could be committed by contractor though, cost is borne by developer. A lack of adequate manpower prevents contractor from completing tasks by 1.79%. It also caused decreasing working hours and productivities. Amendments of drawing increase in cost of work and wasting productive time by 1.61%, consultant aware their responsibilities and there a must to follow government agencies rules in design specification from time to time.

Inadequate details involved 1.11%, in order to convey the complete concept of the project design and resulting work exposed to greater delays where time consuming due to pending adequate confirmation by particular parties. Partial development plan approval by government agencies causing the slow decision making by developers to provide the results to the contractor. As an example, partial approval given by public authorities regarding the detention pond for surface water management, Bin center specification, road works and sewerage treatment plant with 0.91% weightage. In the life cycle of this project, there will almost always be unexpected problems and questions that crop up.

The improper execution of project management derailed project progress having 0.90%. Failing to have the experienced personnel on site involved 0.67% in the construction phase reflected to the mistake during construction stage. Control of human errors during structural construction analyzed to be the key of prevention. Discrepancies of documents in contract, bill of quantities and many procurements linking to the delay in small weightage, 0.23% then again cause reduces the effectiveness of work and re-negotiations among particular parties is taking some time. Public complaints for non-critical issue concerning on environmental impact are properly solved and government authority enforcement dissolved with closed consultation with specialist have shown good results and basically are not minoring cause in delay by which 0.01% respectively.

The physical environment within which a construction project is sited may impact considerably on its development as construction projects are always affected by physical influences [18]. Table 2 shows one of the major delays causes in project. Late monthly progress payment vitally alarming at that time. Developer is taking time to proceed the monthly progress payment with five consecutive months building work interim are delay. Issues over late or withheld payments have touched all parts of the construction especially contractor margin, revolving income and settlement of claims from particulars parties under main contractor such as sub-contractor and supplier. Late and non-payment can create cash flow problems, stress and financial hardship on the contractors and that some reactions to late and non-payment adopted by the contractors may have adverse effects on their own businesses [19].

Figure 1 shows the delay of progress payments against day. In real situation, contractor faced with major pocket account problems in running this project when delay of payments is not solved immediately. The effect of late payments obviously gives impact to the progress with greater weightage. In some cases, contractor has given and had to absorb the cost by late payments to the supplier particularly. Contractor under agreements or any amounts not paid within 30 days from the date of the demand for payment will bear interest. Hence, late payment is a predicament
which is difficult to be dealt with due to different interests of the parties involved [20]. In relation to advancing or borrowing additional capital to fund cost overruns, there will be an increment in interest cost in collecting on another defaulted promise [21].

<table>
<thead>
<tr>
<th>No.</th>
<th>Causes of Delay</th>
<th>Particular</th>
<th>Frequency</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slow decision making</td>
<td>Client</td>
<td>13</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Consultant</td>
<td>27</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Contractor</td>
<td>21</td>
<td>1.01</td>
</tr>
<tr>
<td>2</td>
<td>Mistake during construction stage</td>
<td>Contractor</td>
<td>9</td>
<td>0.67</td>
</tr>
<tr>
<td>3</td>
<td>Discrepancies of documents</td>
<td>Developer</td>
<td>4</td>
<td>0.23</td>
</tr>
<tr>
<td>4</td>
<td>Discrepancies of drawing and specifications</td>
<td>Consultant</td>
<td>22</td>
<td>2.04</td>
</tr>
<tr>
<td>5</td>
<td>Bad weather and force majeure</td>
<td>Nature</td>
<td>443 (days)</td>
<td>3.74</td>
</tr>
<tr>
<td>6</td>
<td>Amendments of drawing</td>
<td>Consultant</td>
<td>31</td>
<td>1.61</td>
</tr>
<tr>
<td>7</td>
<td>Variation order</td>
<td>Developer</td>
<td>41</td>
<td>1.91</td>
</tr>
<tr>
<td>8</td>
<td>Lack of manpower and resources</td>
<td>Contractor</td>
<td>39</td>
<td>1.79</td>
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<td>9</td>
<td>Shortage of skills worker</td>
<td>Contractor</td>
<td>47</td>
<td>2.76</td>
</tr>
<tr>
<td>10</td>
<td>Late delivery from earth work contractor</td>
<td>Developer</td>
<td>1</td>
<td>2.22</td>
</tr>
<tr>
<td>11</td>
<td>Late payment received</td>
<td>Developer</td>
<td>5</td>
<td>2.05</td>
</tr>
<tr>
<td>12</td>
<td>Partial development plan approval</td>
<td>Developer</td>
<td>3</td>
<td>0.91</td>
</tr>
<tr>
<td>13</td>
<td>Inadequate details</td>
<td>Consultant</td>
<td>19</td>
<td>1.11</td>
</tr>
<tr>
<td>14</td>
<td>Project management problem</td>
<td>Contractor</td>
<td>11</td>
<td>0.90</td>
</tr>
<tr>
<td>15</td>
<td>Public complaints</td>
<td>Contractor</td>
<td>5</td>
<td>0.01</td>
</tr>
<tr>
<td>16</td>
<td>Government authority enforcement</td>
<td>Contractor</td>
<td>1</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Total Percent of Delay 25.22

<table>
<thead>
<tr>
<th>Item</th>
<th>Building Work Interim (Month/year)</th>
<th>No. of Days Delay</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No. 13</td>
<td>51</td>
</tr>
<tr>
<td>2</td>
<td>No. 14</td>
<td>36</td>
</tr>
<tr>
<td>3</td>
<td>No. 15</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>No. 16</td>
<td>40</td>
</tr>
<tr>
<td>5</td>
<td>No. 17</td>
<td>12</td>
</tr>
</tbody>
</table>

From different perspective, there are many parties involved being a major cause problem. Payments, which implies a major problem as monies, is needed to pay for materials, labour, plant, subcontractors’ account rendered, preliminaries and general overheads expended during the progress of the work [22]. In every detail of linking to the project, late payment affects personnel and labour on site. When there is late payment, there is a chance to labour quit from working and contractor facing with shortage manpower. In some cases, labour strike and rarely occurred. Referred to the Figure 1, developer late of payments has been delay for almost 2 months in certain interim. The longest late payments period in the interim number 13 with 51 days’ delay. Secondly with interim number 15 with 50 days, interim number 16 within 40 days and interim number 14 are 36 days and interim number 17, delay in days started from interim submission date to client. For overall delay, these issues contributed 2.05% weightage. Developer has acknowledged this issue and point takings in considering contractor’s extension of time application.
To put it another way, late payment received is also a similar issue and problem discussed by other researchers as a delay factor. Delay in settlement of contractor claims are believed is a resultant of payment delays from owners. Contractor or sub-contractors are in the challenge of managing stressed cash flow, contractors are forced to delay payments to vendors and worker wages [27]. Variation order contributed 1.91% to the delay percentage in the project. Variation orders or scope changes resulting in significant changes especially time and sources. In fact, the variation orders issued during construction are major causes of time and cost overruns. Weather grouped as external-related factors. Unfortunately, in this case study, the said project was delay for 443 days or equal to 3.74%. This factor argued as a major contributor to the delay. Compared to the causes of delay to public infrastructure projects [30], weather remained as indisputable factor. In the Malaysian construction industry and Sabah in general, financial problems, poor management, consultant’s supervision problems, consultant’s incompetency and lack of materials in the market were the main delay causes in projects [31].

Figure 2 shows the monthly rainfall analysis for two years starting from the date of commencement in the construction weeks’ period. Contractor in their obligation prepared the weather record that proven by government agencies where there are 443 days of rainy day within 26 months of construction period. Acts of God and Force Majeure Clauses become a claimable reason, where indisputable the extent of weather-related time extensions. Severe weather conditions can be disruptive to construction. Contractors typically obtain time extensions for weather days beyond normal conditions [23]. This situation reinforces the main reasons for the extension of time application’s approval.
4. S-curve Analysis

Contractor has won the work from the consultants and agreed by client in 2012. Contractor ensure the project is built to the agreed quality, budget and timeframe. This project an initial completion within 26-month period from an agreed start date. Unfortunately, project has triggered a breakpoint during the construction period. Regrettably due to unforeseen circumstances contractor have had to extend this event. Contractor are rearranging the amendment completion date so that they can deliver the project in appropriate execution. By applying extension of time, contractor had been to award a full extension of time claim for an extension of time for practical completion. In this case-study, a first stage of postmortem to the initial S-Curve has been identified. Conflict, causes and effect of delay have been discussed in previous section where, the problem identified and dissolved by follow up closely. The initial analysis of S-curve brings the dramatic break point to depict a clear picture this were considered sick project.

In this breakpoint events dynamically changing along the path of project progress and growth before maturation stage achieved. As shown in Figure 3, the initial project S-curve growth progressively up to the first two months from April to May 2013. This project supposed to start from January 2013, due to the late handover from earth work contractor to main contractor, the preliminary works began from April and behind schedule three months. At the first breakpoint, the actual weightage slightly decreases from 5.35% on schedule to delay 4.77%, 0.58% difference. Previously, the movement positively growth 1.56% ahead from schedule. The following months S-Curved planned are updated in line with monthly interim claim throughout the period of the project. Unfortunately, who would have thought that these updates based on financial weightage kept shown decreasing trend until a cumulative delay 5.69% or determined behind schedule up to the 21 months without positive signs. From time to time, contractor has been advised to put more on effort to catch up the delay. December 2014, 21 months’ time elapsed and all particular parties has decided to meet and work out to sort the delay. Contractually, there are still have 5 months to go and seemed all effort are useless without extension of time. In terms of quality and appropriate execution, it has been decided that, contractor should apply for the extension of time. Approved, conditionally in terms of mandatory completion.

From Figure 3, it can be seen that the actual weightage is behind than allocated in schedule. Figure 3 illustrate the changes and shortfalls in monthly progress. It can be clearly seen in January 2014 where, the actual percentage followed closely to schedule with increment 1.42% with overall delay 3.83%. The mean variance is closely monitored, but seemed fail to achieve due to the cumulative work yet to be done. Contractor without intention of deliberately to delay the progress. From Figure 3 initial project S-Curve itself, it is clear that the actual S-curve is far behind the schedule S-curve. There are countless things to learn from some practicality of S-curve figure as following.

- Concurrent delay occurs;
- Poor project management;
- Inappropriate work sequence;
- Double handling;
- Unsystematically time management;
- Priority work is yet to begin.

However, the identified lesson learned a picture of situation, the real condition of causes and effect as per discussed in previous section. An overview of project are governed by many factors. Quality is a phenomenon, it is an emergent property of people’s different attitudes and beliefs, which often change over the development life-cycle of a project [24]. The initial S-curve has been modified to match the approval 6 months’ additional time of extension. By eliminating high steep and any lope hole from commissioning the rebound phase, process to modify the S-curve and introducing turning point by adding 6 months’ time of extension starts with sorting the high delay weightage to minor activities. The outstanding problem previously identified. This Figure 5, modified project S-curve has been projected as accumulated data from the monthly progress claim since this S-curve are generated from financial progress claim.

In term of time frame based on the projected S-curve accordingly to extension of time, the turning point of work progress moved along to the adjacent stride atwards time. This means that, contractor has to emphasize the bigger delay percentage and concurrently finish the small variance of remaining work. To uplift the growth of progress, project management have committed to the time given. Figure 4 shows the adjustment S-curve figure where, 32 months have been projected with rebound percentage strengthening the monthly progress by eliminating delay factors. In that events, the break point from delay to catch up the schedule broke the trigger in July 2014 where, the percentage reportedly increase against time. This is where, monitoring skills from S-curve has been pictured and control closely. Again, contractor reportedly almost broke the events to behind schedule where actual percentage drop significantly in July 2015, 4 months before the dead line. Problem identified immediately and dissolved due to unexpected material late delivery from market disruptions that resulted in reduction percentage. Contractor carefully planned the work,
resulted in good achievement in remaining months to go. As can be seen, at 30th month, contractor has achieved almost 97% of work, this is where the time line has achieved success, where the remaining 3% for provisional sum.

Figure 3. Initial Project S-Curve

Figure 4. Modified S-Curve with extension of time (EOT)

An understanding of S-curve practically and its analyses will help contractor and particular parties grasp the importance of monitoring the progress and growth of an ongoing project. By generating a S-curve comparison of baseline to actual values, contractor or site personnel can evaluate the accuracy of cost or work estimates used to approve projects. However, the matter of understanding the significance of its practically and its analyses is of utmost importance. Figure 4 depicts the following brief of modified S-curve in this case-study’s project, as important information related to the modification factor. Moreover, weather also known as the major relative factor in the mentioned area due to climate change that are manifested by changes in temperature, precipitation and the annual mean rainfall is 3599 mm and the timing of peak month in May and November [39].

5. Contractor Procurement

Information disclosure throughout the procurement process and application of the extension of time must be thorough and careful preparation. In this case-study’s project, during the construction process contractors to undertake obligations with caution and ensure everything is in satisfactory writing. From the delay causes, late payment has recorded as the contributor to the weightage of delay with almost 51 days. Contractor in written notice reminding client that they had the obligation to delay intentionally effects of late payment. All procurements must be in presentable and written. These documentations a later become supplementary procurements in order to apply an
extension of time. Although it is difficult to measure the delays on the baseline program updates this will help the contractor to ease out some of the problems until the revised programs are approved [8]. Proactive measures include quick responses to complaints from clients, requesting written confirmation on any important verbal conversation or instruction, extension of time requests on excusable delays, records on any disagreements that arise with clients or his representatives, and clarification on any instruction or change order prior to the commencement of such extra works [25].

Table 3. Comparison of finding in this study with causes of delay in other selected studies

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<tbody>
<tr>
<td>1</td>
<td>Slow decision making</td>
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<td>1</td>
<td>1</td>
<td>5</td>
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<td>2</td>
<td>Mistake during construction stage</td>
<td>1</td>
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<td>3</td>
<td>Discrepancies of documents</td>
<td>2</td>
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<td>4</td>
<td>Discrepancies of drawing and specifications</td>
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<td>5</td>
<td>Bad weather and force majeure</td>
<td>1</td>
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<td>6</td>
<td>Amendments of drawing</td>
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<td>8</td>
<td>Lack of manpower and resources</td>
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<td>9</td>
<td>Shortage of skills worker</td>
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<td>1</td>
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<td>10</td>
<td>Late delivery from earth work contractor</td>
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<td>11</td>
<td>Late payment received</td>
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<td>4</td>
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<td>12</td>
<td>Partial development plan approval</td>
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<td></td>
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<td>3</td>
<td>2</td>
<td>1</td>
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<td>13</td>
<td>Inadequate details</td>
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<td>1</td>
<td>1</td>
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<td>Project management problem</td>
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<td>16</td>
<td>Government authority enforcement</td>
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<td>8</td>
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</table>

Table 3 provides the comparison of statistical measures and discussion about the delay factors from various researchers. Collectively, from the documentations and respondent information’s, this study has determined 16 most important causes of delay. From Table 3, these results reveal a strong agreement between each factor and each project history. The causes of delay are common problem and factors. In general, this study has identified various factors of delay contributes by all parties. Comparison with previous studies was done from various countries, 4 from Malaysia, Saudia Arabia, Egypt, Turkey and Jordan each respectively. Seven related studies were compared, to an extent, there are temporal commonalities with the work reported herein which are, the 16 causes that discussed in this study is similar and most important factors that happened to various project background in general. Thus, it can be concluded that, there are 5 groups that contributed to the delay factor described as follows:

- a) Owner-related factors;
- b) Consultant-related factors;
- c) Designer-related factors;
- d) Contractor-related factors;
- e) Labor-related factors;
- f) External-related factors.

6. Conclusion

This presentation is a case-study regarding the analysis of S-Curve of a life project draws comparative interpretation of project performance towards project delivery schedule. Based on case-study’s history, there are some conclusion can be made through the S-curve of delay project. The remedial and performing a good practice in construction to eliminate delay to catch up progress. Markedly, risk and break point delay are viewed as lessons learned. Significantly, the S-Curves based on financial are applicable to project management applications in this situation. Costs versus Time S-Curve be a baseline in monitoring project progress. Consequently, S-Curve representing the standard distribution of costs over time and a cumulative flow of money over a time period. The
optimization process of second phase in construction during extension of time are greatly monitored using the S-Curve and doesn’t change the relationship between activities. Notwithstanding, document review and checking should always be under the responsibility of contractor and should reporting any discrepancies to the architect or engineer be mandatory. In the final analysis, failing to have the experienced personnel on site involved reflected to the mistake during construction stage. Eventually, in term of time frame based on the projected S-curve accordingly to extension of time, the turning point of work progress moved along to the adjacent stride towards time. This study encapsulates the risk factor and document analysis strategies, which enable construction firms and to prepare the required document for EOT application and evaluate the risk to deliver construction projects on time and within a scheduled budget.

7. Declarations

7.1. Data Availability Statement

The data presented in this study are available in article.

7.2. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

7.3. Acknowledgements

The authors would like to acknowledge support from the particular universities and company in which the author performing his duties especially Universiti Malaysia Sabah. Special gratitude is given to those industry practitioners who responded and contributed in this case-study project. This work was financially supported by Universiti Malaysia Sabah (UMS), Research and Innovation Management Centre.

7.4. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Development of a Methodology for Assessing the Technical Level of Cultural Heritage Objects in Construction

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Received 15 January 2021; Revised 20 March 2021; Accepted 24 March 2021; Published 01 April 2021

Abstract

Cultural objects (building and construction) that are considered as the most valuable segment of real estate in terms of historical heritage usually evaluated only by monetary aspects. Not concreteness of the approaches to the determination of the historical heritage objects leads to the fact that its original value often remains unread, the assessment has subjective character, reflects the values of certain time frame and place. Therefore, for the purpose of determination of value of the historical heritage objects, there is need for the creation of the unified classification system of their assessment. Thus, this research is considered to be very relevant. The paper investigates intangible factors that affect the evaluation of cultural heritage objects in construction. In the work a system of value indicators, which take into account not only tangible indicators but also the intangible value of cultural heritage objects, has been developed. Developed indicators system makes it possible to evaluate the historical and cultural value of real estate objects quite objectively on the contrary to common methods which deal only with monetary aspects of market price of such objects. A methodology for a comprehensive evaluation of cultural heritage objects has been formed as well. This methodology is based on both the traditional comparative approach and the system of value indicators. As a result, suggested integrated approach has been proven to provide fair evaluation of both the tangible and intangible characteristics and improve the quality of cultural heritage objects assessment process.

Keywords: Cultural Heritage Objects; Planning System; Intangible Value; Market Evaluation.

1. Introduction

Cultural heritage objects (building and construction) are the most valuable segment of real estate in terms of history, archeology, architecture, urban planning, art, science and technology, aesthetics, ethnology or anthropology, social culture and are evidence of eras and civilizations, sources of information about culture [1].

Despite the value of cultural heritage objects, most of these constructions and buildings require restoration. Therefore, there is a practice of transferring cultural heritage objects into the ownership of investors, or in transferring objects for a long-term lease (usually for 49 years). Sale or long-term lease of cultural heritage sites are one of the ways to preserve the intangible value of such buildings [2].

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http://dx.doi.org/10.28991/cej-2021-03091680

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As a result, cultural heritage objects are involved into market turnover. For the purposes of selling, insuring, privatizing, pledging in a bank, an assessment of these objects is required. A contradiction arises between the need to conduct market transactions with cultural heritage objects and the lack of an objective methodology for evaluation of the intangible component that present historical and cultural value of these objects to the real estate market.

A comprehensive methodology for assessing design solutions for cultural heritage objects is necessary not only for performing operations with these objects (sale, privatization, insurance, long-term lease), but also for presenting their historical, cultural, architectural and aesthetic value [3].

Traditional approaches to real estate market value assessment usually take into account only the technical characteristics of buildings, ignoring their historical and cultural value. The Methodological Recommendations for the Assessment of Real Estate Objects (dated 23.06.2015) states that the intangible value should be taken into account via special adjustment factors. However, sufficient and thorough researches on the development of a system of value indicators can be hardly found.

The lack of objective approaches to the cultural heritage objects evaluation leads to inadequate, high subjective and untrustworthy results. Thus, the problem of cultural heritage objects evaluation is relevant and requires the development of a value indicators system. The purpose of the study is to improve the quality of the cultural heritage objects assessment process based on the development of a value indicator system. Research objects is assessment process of the technical state of design solution of cultural heritage objects.

The subject of the research is the value indicators system for cultural heritage objects. The scientific novelty of the research lies in the development of a value indicators system that will take into account the intangible value of cultural heritage objects. This system allows obtaining an objective market value of the historical and cultural objects. The practical value comes from development of a comprehensive methodology for evaluation of real estate cultural heritage sites.

To achieve this goal, the following tasks have been solved:

- Research of intangible value of cultural heritage objects;
- Analysis of methods for assessing cultural heritage sites;
- Development of a value indicators system;
- Development of a methodology for evaluation of design solution technical state of cultural heritage objects.

According to the tasks solved the structure of the article includes the following sections:

- Cultural heritage objects classification;
- Comparative analysis of methodologies for cultural heritage objects evaluation;
- Methods of cultural heritage objects evaluation;
- Characteristics of the indicators system for cultural heritage objects;
- The results of the expert assessment,
- Discussion;
- Conclusion.


2.1. Cultural Heritage Objects Classification

According to the normative and technical documents, the assessment of the design solution technical state of capital construction projects is a set of operations (measures) aimed at comparing the technical and economic indicators and quality characteristics of analyzed object with baseline of these indicators. Evaluation of the design solutions technical state of cultural heritage objects has its own specifics, the quality indicators of these objects have historical, cultural, architectural and aesthetic value, which is absent in the typical real estate objects [3].

It should be noted that cultural heritage objects (monuments) can be profitable on its own or financed from the city/region/state budget [4]. This work considers only profitable objects. Result analysis of scientific information search allowed building a multilevel classification of cultural heritage objects (Table 1).
Table 1. Classification of cultural heritage objects

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Object examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Types</td>
<td>monuments, ensembles, sights</td>
</tr>
<tr>
<td>Category</td>
<td>objects of federal level, objects of regional level, objects of municipal level</td>
</tr>
<tr>
<td>Profitability</td>
<td>profitable on their own, financed from state budget</td>
</tr>
<tr>
<td>Type of design solution</td>
<td>enfilade, cabinet, enfilade-cabinet, compact (centric)</td>
</tr>
</tbody>
</table>

Types of design solutions have its own features, advantages and disadvantages:

- Enfilade type is a building planning system that provides for the transition from one room to another through openings in walls or partitions. The system does not include corridors and other areas that reduce the usable area of the building;
- Cabinet type is a system, most commonly used in office buildings, involves the location of rooms on both sides of the corridor;
- Enfilade-cabinet type is a combination of the first two systems;
- Compact (centric) type is characterized by a large room in the center (core) with surrounding rooms of a smaller area.

Result analysis of scientific information search allowed building a multilevel classification of cultural heritage objects (Table 2).

Table 2. Advantages and disadvantages of design solution types

<table>
<thead>
<tr>
<th>Design solution type</th>
<th>Advantages</th>
<th>Advantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enfilade</td>
<td>- Buildings have sufficient bearing capacity</td>
<td>- A large number of walk-through rooms that reduces usable area</td>
</tr>
<tr>
<td></td>
<td>- Ability to re-plan during reconstruction</td>
<td>- The planning system is suitable only for objects with museum/exhibition purposes, but not for profitable real estate</td>
</tr>
<tr>
<td></td>
<td>- Ability to designing compact buildings</td>
<td>- Lack of utility rooms</td>
</tr>
<tr>
<td>Cabinet</td>
<td>- System provides private spaces and rooms</td>
<td>- Inefficient usage of office space</td>
</tr>
<tr>
<td></td>
<td>- Minimization of noise level</td>
<td>- Additional expenses for lighting, air conditioning</td>
</tr>
<tr>
<td>Enfilade-cabinet</td>
<td>- Efficient usage of spaces</td>
<td>- Long dark corridors through the building</td>
</tr>
<tr>
<td></td>
<td>- Usage of advantages of both solution types</td>
<td></td>
</tr>
<tr>
<td>Compact (centric)</td>
<td>- Suitable for cultural spaces like cinemas, concert halls, theatres</td>
<td>- Inefficient in terms of profitability (such buildings are hard to lease)</td>
</tr>
</tbody>
</table>

It is clear from the table that the most suitable design solution type for profitable real estate objects is enfilade-cabinet one.

2.2. Comparative Analysis of Methodologies for Cultural Heritage Objects Evaluation

The following diagram (Figure 1) can represent the main approaches in the field of preservation and protection of cultural heritage objects.

![Figure 1. Main approaches for preserving cultural heritage objects worldwide](image-url)
Analysis of the policies in the field of protection and preservation of cultural heritage objects in foreign countries [5-7], the following conclusions have been drawn:

- The preservation and maintenance of the cultural heritage objects is impossible only with the funds of the state budget [8-10];
- In most countries, assessment and evaluation of cultural heritage objects are carried out by special committees [15-18];
- Attraction of investments for the protection, restoration and preservation of cultural heritage objects can be done in various ways (privatization, sale, trust management, long-term lease, and tax incentives) [12-14].

2.3. Methods of Cultural Heritage Objects Evaluation

In European countries, there are several methods for evaluation of cultural heritage objects. The most spread ways to use privatized cultural heritage objects are associated with tourism: museum sites, recreational sites (hotel use, residential real estate open for visits and excursions) [11, 12]. The most popular evaluation methods come from the UK and undergo continuous improvement even today. The diagram (Figure 2) shows the main methods for evaluation of cultural heritage objects in European countries [17].

![Figure 2. Methods for evaluation of cultural heritage objects in European countries](image)

The travel cost method is to convert the time and money spent on visiting a cultural heritage object into characteristics of cultural heritage object (technical state) [14, 15]. The technical state of a cultural heritage property is calculated as the sum of the cost per visitor of this property for tickets, fuel and time multiplied by the average annual number of visitors. The scheme for assessing the technical level of design solutions is shown in Figure 3 [17, 18].

![Figure 3. Travel cost method scheme](image)

Monetary values for time and fuel can be derived from Department of Transportation data (Forrest et al. 2000). The calculations use averaged data. The technical level of design solutions, calculated by the travel costs method, can be represented by the following formula [19]:

\[
TL = (C_e + C_t + C_f) \times n
\]

Where TL – Technical level of cultural heritage objects, \(C_e\) is entry ticket cost, \(C_t\) is monetary value of time spent on travelling and site visit, \(C_f\) is fuel cost spent on travelling to the site, \(n\) is the number of annual visitors.

The main disadvantages of the travel cost method are:

- The travel time and cost cannot accurately reflect the technical level of the property itself;
- In order to adequately determine the cost of time and fuel, it is necessary to conduct consumer surveys, which leads to errors in calculations;
This method is applicable only for a narrow segment of cultural heritage objects (such as museums, cultural spaces etc.).

This method was widely used to evaluate Scottish museums, objects in Armenia, as well as the Alto Douro wine region in Portugal. The contingent evaluation method is a universal and commonly used in European countries. It is based on finding the average technical level of the object that include consumer and unused value [15, 18].

The consumer value is formed for profitable real estate objects and is determined by the market price of similar objects. This cost does not take into account the cultural value of the object and reflects only the economic component [17]. The unused value shows the intangible value of an object and as well, as how much a country's residents are willing to pay to preserve this object and pass on its heritage to the next generations. The expert group draws up questionnaires and, based on the results of a survey of country residents, release a coefficient of the unused value. This method cannot be called accurate, since the survey answers may be biased.

The hedonic pricing method is based on intangible factors that increase the value of a cultural heritage property. Researches reveal the dependence of sale price of the objects located near cultural heritage objects in relation to the sale price of similar real estate objects located in an ordinary city district. A calculated coefficient reflects the value of the cultural heritage object [16, 17]. Also in the UK, there is an expert method, in which independent specialists (experts) derive coefficients reflecting the intangible value of an object. The coefficients were calculated according to the following criteria: the level of aesthetics, spiritual value, social value, authorship, the symbol of the settlement, the historical symbol, the historical value [21, 22]. There are several standard approaches to real estate evaluation in Russian Federation [13, 23-26] (Figure 4).

**Figure 4. Descriptive scheme for standard approaches to real estate evaluation in Russian Federation**

Main disadvantages of evaluation methods mentioned above are presented in Figure 5.

**Figure 5. Descriptive scheme for standard approaches to real estate evaluation in Russian Federation**

The traditional methods, provided in Figure 4 are not applicable in their classical treatment to assessment of technological level of objects of cultural heritage. Therefore, the author's techniques, which are briefly analyzed in the article, are in practice known. T.A. Slavina's technique is the earliest of all the techniques existing for now. It is based on the cost approach, considers some technical and quality characteristics of the building, without the land plot under the objects. The technological level of objects of cultural heritage (according to Slavina's technique is presented in the general form in Figure 6.
A.V. Lukov’s technique is also based on the cost approach. The calculation is conducted without the cost of the land plot under the object of cultural heritage. The technique can be in general expressed by the Equation 2:

$$T = R_s \times (1 + C_o) \times C_u \times (1 - W) - ZDP$$  \hspace{1cm} (2)

Where: $T$: Technological level of the object of cultural heritage; $R_s$: Recovery size (cost of new construction, cost of restoration and residual cost); $C_o$: Dimensionless coefficient, building value indicator; $C_u$: Uniqueness coefficient; $W$: Saved-up wear; ACM: The additional costs for the object maintenance.

E.E. Yaskevich’s technique is based on two methods, which are the cost and revenue methods:

$$T = R_s \times PP \times (1 - W_e) + P_{If} - S_s + C_{HZ} \pm HBC$$  \hspace{1cm} (3)

Where: $T$: Technological level of the object of cultural heritage; $R_s$: Recovery size; $PP$: Entrepreneur profit; $W_e$: External wear; $P$: Prestigiousness; $P$: Physical deterioration; $I$: Easement size; $S_s$: The size of the land plot; $C_{HZ}$: Specific features; HBC: Unrecorded value types.

However, it is difficult to determine the size of prestigiousness of the object with the use of this technique. O.E. Tolstova's technique assumes the assessment of object of cultural heritage as the standard building which is not of cultural value. This technique is based on the comparative and revenue methods. The income analysis which brings the object of cultural heritage and the standard building is made. On the basis of the obtained information "the historical reputation" of the object is defined. However, some difficulties of practical application of this technique are caused by closeness of information on the market of objects of cultural heritage.

The author's technique created by Bashkatov V.S. which is based on the cost approach is the most clear one. According to this technique the technological level of the object of cultural heritage consists of three composed, including the land plot, improvements and non-material component. The non-material component is defined by assessment of labor input of works on restoration of architectural decorative elements. Having carried out the analysis of the aforesaid author's techniques, we should note that the well-known author's techniques have the problem of the translation of quality culturological characteristics of the object of cultural heritage into quantities. Besides, the cost approach does not allow to define the non-material component of the object.

In practice, appraisers adapt standard approaches when evaluating a specific property. Thus, there is a clear need in a unified methodology for real estate evaluation, which would take into account the intangible value of cultural heritage objects. Moreover, international experience in evaluation of cultural heritage objects does not have the practice of re-profiling cultural heritage sites for other purposes.
3. Research Methods: Development of a Value Indicators System for Cultural Heritage Objects

According to the analysis of the evaluation methods of the objects of cultural heritage (item 2) and techniques of the assessment of objects of antiques and art products, we can draw the conclusion about the impossibility of application of the cost approach to the assessment of objects of cultural heritage. According to the authors, it is possible to apply the method of the analysis of hierarchies which is used for the assessment of art products and products from antiques to the assessment of objects of cultural heritage. The essence of the method of the analysis of hierarchies is provided in figure 7 (the global purpose is the object assessment; the intermediate levels (criteria) are the characteristics of the object; alternatives are the similar objects).

![Figure 7. Stages of the method of the analysis of hierarchies](image)

To develop a system of value indicators, cultural heritage objects are divided by profitability into several groups as following:

- Buildings fully equipped for commercial purposes (offices, banks, residential and hotel real estate),
- Buildings partially equipped for commercial purposes,
- Buildings not meant for commercial use (culture, art, education),
- Religious buildings,
- Public buildings (consulates, budgetary organizations, government buildings),
- Buildings in disrepair, the use of which is impossible without proper restoration.

Objects of cultural heritage have intangible value, which is determined by a set of certain factors (Table 3) [27, 29].

<table>
<thead>
<tr>
<th>Value type</th>
<th>Advantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Historical</td>
<td>Date of construction; Monument status; Relation to a certain architectural style and epoch; Connection to certain historical events or personalities; Ownership history.</td>
</tr>
<tr>
<td>Urban</td>
<td>The role of the object in the urban system or the natural environment; Increase of the attractiveness of the location due to the presence of an object; Stimulation of building constructions of architectural style of an object.</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>Relation to the heritage of a certain author; Relation to a certain architectural ensemble; The architectural uniqueness of an object; The architectural appearance of the building facades.</td>
</tr>
<tr>
<td>Utilitarian</td>
<td>The degree of originality of the architectural and constructive solution; Technological features; The role of the object’s architect in art history; Degree of object’s preservation.</td>
</tr>
</tbody>
</table>
To determine the influence degree of factors from Table 3 on the intangible value of cultural heritage objects, the method of hierarchy analysis [28, 30] has been used and a matrix of priority criteria has been built (Table 4). If, when comparing one factor \(i\) with another \(j\), \(a_{(i, j)} = b\) is obtained, then when comparing the second factor with the first, \(a_{(j, i)} = 1/b\) is obtained. The more important the criterion is, the more integer scores there will be in the corresponding row of the matrix, and the scores have greater values. Next, the geometric mean in each row of the matrix is calculated:

\[ a_i = \sqrt[n]{\text{product of elements in a row } i} \] (4)

Geometric means are calculated as:

\[ \sum a_i = a_1 + a_2 + \cdots + a_n \] (5)

Calculation of the normalized priority vector:

\[ \text{component } i = \frac{a_i}{\sum a_n} \] (6)

When compiling a matrix of priority criteria, the expert must answer several questions: “Which one is more important and has the greatest impact?” and “Which one is preferable and has the highest probability?”.

4. Results and Discussion

According to the results of the expert assessment, the priority weight of the criteria was calculated (Table 4).

<table>
<thead>
<tr>
<th>Criterion/Value</th>
<th>Historical</th>
<th>Urban</th>
<th>Aesthetic</th>
<th>Utilitarian</th>
<th>(II<em>III</em>IV*V)/4</th>
<th>Criterion weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
<td>VI</td>
<td>VII</td>
<td></td>
</tr>
<tr>
<td>Historical</td>
<td>1</td>
<td>3</td>
<td>1/3</td>
<td>3</td>
<td>1,31607</td>
<td>0,3170</td>
</tr>
<tr>
<td>Urban</td>
<td>1/3</td>
<td>1</td>
<td>3</td>
<td>1/3</td>
<td>0,75984</td>
<td>0,1830</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>1/3</td>
<td>3</td>
<td>1</td>
<td>1/3</td>
<td>0,75984</td>
<td>0,1830</td>
</tr>
<tr>
<td>Utilitarian</td>
<td>1/3</td>
<td>3</td>
<td>3</td>
<td>1</td>
<td>1,31607</td>
<td>0,3170</td>
</tr>
<tr>
<td>Priority matrix of an cultural heritage appraiser</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4,15182</td>
<td>1,0000</td>
</tr>
<tr>
<td>Historical</td>
<td>1</td>
<td>3</td>
<td>1/3</td>
<td>1/3</td>
<td>0,0833</td>
<td>0,0481</td>
</tr>
<tr>
<td>Urban</td>
<td>1/3</td>
<td>1</td>
<td>5</td>
<td>3</td>
<td>1,2500</td>
<td>0,7212</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>3</td>
<td>1/5</td>
<td>1</td>
<td>1</td>
<td>0,1500</td>
<td>0,0865</td>
</tr>
<tr>
<td>Utilitarian</td>
<td>3</td>
<td>1/3</td>
<td>1</td>
<td>1</td>
<td>0,2500</td>
<td>0,1442</td>
</tr>
<tr>
<td>Priority matrix of an architect</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,7333</td>
<td>1,0000</td>
</tr>
<tr>
<td>Historical</td>
<td>1</td>
<td>3</td>
<td>1/3</td>
<td>1/3</td>
<td>0,0833</td>
<td>0,0268</td>
</tr>
<tr>
<td>Urban</td>
<td>1/3</td>
<td>1</td>
<td>1/3</td>
<td>1</td>
<td>0,0278</td>
<td>0,0089</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>3</td>
<td>3</td>
<td>1</td>
<td>1/3</td>
<td>0,7500</td>
<td>0,2411</td>
</tr>
<tr>
<td>Utilitarian</td>
<td>3</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>2,2500</td>
<td>0,7232</td>
</tr>
<tr>
<td>Priority matrix of an real estate appraiser</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3,1111</td>
<td>1,0000</td>
</tr>
<tr>
<td>Historical</td>
<td>1</td>
<td>1/2</td>
<td>3</td>
<td>3</td>
<td>1,1250</td>
<td>0,6279</td>
</tr>
<tr>
<td>Urban</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1/2</td>
<td>0,5000</td>
<td>0,2791</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>1/3</td>
<td>1/2</td>
<td>1</td>
<td>2</td>
<td>0,0833</td>
<td>0,0465</td>
</tr>
<tr>
<td>Utilitarian</td>
<td>1/3</td>
<td>2</td>
<td>1/2</td>
<td>1</td>
<td>0,0833</td>
<td>0,0465</td>
</tr>
<tr>
<td>Priority matrix of manager in historical restoration projects</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,7917</td>
<td>1,0000</td>
</tr>
</tbody>
</table>

Based on the results of the expert assessment, the final priority matrix has been formed (Table 5).
Table 5. Priority matrix based on experts’ evaluation

<table>
<thead>
<tr>
<th>Value indicator</th>
<th>Historical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Historical value</td>
<td>0.2550</td>
</tr>
<tr>
<td>Urban value</td>
<td>0.2980</td>
</tr>
<tr>
<td>Aesthetic value</td>
<td>0.1393</td>
</tr>
<tr>
<td>Utilitarian value</td>
<td>0.3077</td>
</tr>
</tbody>
</table>

As it can be seen that the utilitarian value has the highest weight, urban value is on the second place, and historical and aesthetic values share third and fourth places correspondingly. The developed value indicators system (Table 5) is accompanied by a methodology for a comprehensive evaluation of cultural heritage objects. This methodology is based on a comparative approach with comparison between evaluated objects and similar ones.

Suggested methodology for a comprehensive evaluation of cultural heritage objects includes the following stages:

- Market research and selection of listings (offers for sale) of real estate analogs;
- Collection and verification of information on each analogue object;
- Comparison of each analogue object with the evaluated object via pricing parameters;
- Correction of listing prices for each analogue object with consideration of differences between them and the evaluated object;
- Alignment of the adjusted prices of analog objects;
- Adding the intangible value via value indicators system and the hierarchy analysis method (Table 4).

The non-material value of the objects is estimated by the system of valuable indicators (Table 4) on the basis of method of the analysis of hierarchies. As a result of comparative analysis the similar objects are revealed at the technological level expressed in the corrected cost without non-material factors. The first stage of the method of the analysis of hierarchy is the structuring problem of calculation in the form of hierarchy.

The second stage is the comparison of extent of the factors (historical, town-planning, esthetic and utilitarian value) influence on the object of the assessment and the similar-objects according to the scale of relative importance. The total matrix of comparison is formed.

Thus, because of the comparative approach different analogous objects with the technical level of design solutions, expressed in adjusted cost, excluding intangible factors can be considered. Intangible factors are taken into account using the hierarchy analysis method. Comparison of the influence degree of intangible factors on the evaluated object and analogue objects on a scale of relative importance, allows forming the final matrix (Table 6).
Table 6. Priority matrix based on experts' evaluation

<table>
<thead>
<tr>
<th>Criterion/Value</th>
<th>Historical</th>
<th>Urban</th>
<th>Aesthetic</th>
<th>Utilitarian</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Evaluated object</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>A<em>0.2549+B</em>0.2980+C<em>0.1393+D</em>0.3077</td>
</tr>
<tr>
<td>Analogue object #1</td>
<td>X</td>
<td>Y</td>
<td>Z</td>
<td>Q</td>
<td>X<em>0.2549+Y</em>0.2980+Z<em>0.1393+Q</em>0.3077</td>
</tr>
<tr>
<td>Analogue object #2</td>
<td>J</td>
<td>L</td>
<td>M</td>
<td>N</td>
<td>J<em>0.2549+L</em>0.2980+M<em>0.1393+N</em>0.3077</td>
</tr>
<tr>
<td>Analogue object #3</td>
<td>R</td>
<td>P</td>
<td>S</td>
<td>K</td>
<td>R<em>0.2549+P</em>0.2980+S<em>0.1393+K</em>0.3077</td>
</tr>
</tbody>
</table>

The technological level of the object $T$ is calculated by the following formula:

$$T = \frac{\sum(T_i \cdot K_i)}{(1-K_0)}$$ (7)

Where: $T_i$: Technological level of the i-th object analog; $K_i$: Weight coefficients for the i-th similar object; $K_0$: Weight coefficient for the estimated object.

As the result of the executed research the valuable indicators of objects of cultural heritage are systematized. It is established that the most priority factor is utilitarian value, then we should consider city-planning value, then historical and esthetic value. The main evaluation stages of technological level of objects of cultural heritage are defined. It is established that the combination of comparative approach and method of the analysis of hierarchies allows to estimate non-material factors of objects of cultural heritage objectively and to consider them correctly and to increase the quality of estimated works.

The comparative analysis of the assessment results of technological level of the objects of cultural heritage received on the basis of the developed technique and earlier known techniques and approaches (see item 2) proved the following:

- The use of standard approaches (cost-effective and profitable method, comparative method) did not allow to receive objective assessment of technological level of object as qualitative indexes (non-material values) of objects were not considered;
- In practice experts-appraisers are forced to spend time for correction of standard approaches for each estimated object of cultural heritage;
- Because of big share of subjective adjustments there are discrepancies in assessment of technological level of objects of cultural heritage and disputes with the state and bank checking instances, and assessment procedure of objects drags on for an indefinite term.

5. Conclusions

Practical approbation of the evaluation method of technological level of object of cultural heritage was carried out for the memorial building in St. Petersburg). The fragment of the assessment results of technological level of object is given below. Calculation of valuable indicators was made by the means of the method of the analysis of hierarchy. The object of assessment and similar objects were compared on the following factors: historical, town-planning, esthetic and utilitarian values. The results of comparative analysis of objects by means of scale of relative importance (Table 1), are provided in the priority matrixes (Tables 7 to 10).

Table 7. Priority matrix for the historical value

<table>
<thead>
<tr>
<th>Object of assessment</th>
<th>Similar object #1</th>
<th>Similar object #2</th>
<th>Similar object #3</th>
<th>Similar object #4</th>
<th>Similar object #5</th>
<th>(II<em>III</em>IV+V+VI*VII)^1/6</th>
<th>Criterion weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
<td>VI</td>
<td>VII</td>
<td>VIII</td>
</tr>
<tr>
<td>Object of assessment</td>
<td>1</td>
<td>1</td>
<td>5</td>
<td>3</td>
<td>1/3</td>
<td>2</td>
<td>1.4678</td>
</tr>
<tr>
<td>Similar object #1</td>
<td>1</td>
<td>1</td>
<td>5</td>
<td>3</td>
<td>1/3</td>
<td>2</td>
<td>1.4678</td>
</tr>
<tr>
<td>Similar object #2</td>
<td>1/5</td>
<td>1/5</td>
<td>1</td>
<td>1/3</td>
<td>1/6</td>
<td>1</td>
<td>0.3612</td>
</tr>
<tr>
<td>Similar object #3</td>
<td>1/3</td>
<td>1/3</td>
<td>3</td>
<td>1</td>
<td>1/3</td>
<td>3</td>
<td>0.8327</td>
</tr>
<tr>
<td>Similar object #4</td>
<td>3</td>
<td>3</td>
<td>6</td>
<td>3</td>
<td>1</td>
<td>5</td>
<td>3.0531</td>
</tr>
<tr>
<td>Similar object #5</td>
<td>1/2</td>
<td>1/2</td>
<td>1</td>
<td>1/3</td>
<td>1/5</td>
<td>1</td>
<td>0.5054</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.6880</td>
<td>1.0000</td>
</tr>
</tbody>
</table>
The total matrix of valuable indicators (Table 11) is given below and the technological level of the studied object is calculated by Equation 7.
### Table 11. Total matrix of the studied object

<table>
<thead>
<tr>
<th></th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
<th>VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Object of assessment</td>
<td>0.91</td>
<td>0.130</td>
<td>0.104</td>
<td>0.117</td>
<td>0.138</td>
<td></td>
</tr>
<tr>
<td>Similar object #1</td>
<td>0.191</td>
<td>0.254</td>
<td>0.218</td>
<td>0.260</td>
<td>0.042</td>
<td></td>
</tr>
<tr>
<td>Similar object #2</td>
<td>0.047</td>
<td>0.032</td>
<td>0.033</td>
<td>0.053</td>
<td>0.117</td>
<td></td>
</tr>
<tr>
<td>Similar object #3</td>
<td>0.108</td>
<td>0.153</td>
<td>0.059</td>
<td>0.117</td>
<td>0.117</td>
<td></td>
</tr>
<tr>
<td>Similar object #4</td>
<td>0.397</td>
<td>0.065</td>
<td>0.424</td>
<td>0.382</td>
<td>0.297</td>
<td></td>
</tr>
<tr>
<td>Similar object #5</td>
<td>0.066</td>
<td>0.366</td>
<td>0.162</td>
<td>0.072</td>
<td>0.171</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0000</td>
<td>1.0000</td>
<td>1.0000</td>
<td>1.0010</td>
<td>1.000</td>
<td></td>
</tr>
</tbody>
</table>

Technological level of the estimated object:

$$C = (0.235 \times 362,472 + 0.042 \times 325,829 + 0.117 \times 232,431 + 0.297 \times 306,313 + 0.171 \times 308,643 / (1 - 0.138) = 254,899 \text{ rubles sq. meter}.$$  

The received results of the assessment of technological level of the studied object are recommended for the use at transactions in the real estate market:

- For the purpose of purchase sale/privatization of object;
- For transfer object in lease or pledge;
- For insurance of object;
- For crediting;
- When entering into the authorized capital of the company;
- During business valuation;
- For the purposes of taxation;
- By reorganization and privatization of the enterprise;
- For permission of receivership proceeding;
- For the acceptance of various management decisions (for calculation of cost of restoration of objects of cultural heritage).

Users of the developed technique can include: appraisers of the real estate, appraisers of business, owners of object of cultural heritage, specialists of mortgage department of bank, specialists of insurance companies, developers who are engaged in management of similar objects, construction companies which are engaged in restoration of similar objects.

Based on the results of the study, the main conclusions have been formulated in the following way:

- The value indicators system allows forming an integrated approach to evaluation of the design solution technical level of different cultural heritage objects;
- The developed methodology takes into account the tangible (material) and intangible (historical and cultural) factors of the evaluated object;
- The combination of a comparative approach and the hierarchy analysis method in the evaluation of the design solution technical level allows adequately assessing intangible factors and improve the quality of evaluation process;
- The methodology for a comprehensive evaluation can be applied in the real estate market in various types of transactions.

### 6. Declarations

#### 6.1. Author Contributions

L.L. developed the methodology of the complex assessment of the technological level of objects of cultural heritage on the basis of comparative approach and method of the analysis of hierarchies; S.T. systematized the physical (material) and non-material (historical and cultural) factors of the assessment object, developed the system of
valuable indicators of objects of cultural heritage: L.T. developed the algorithm of the object assessment and the matrix of priority. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

6.4. Conflicts of Interest

The authors declare no conflict of interest.

7. References


An Experimental Study on Concrete’s Durability and Mechanical Characteristics Subjected to Different Curing Regimes

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Received 08 January 2021; Revised 21 March 2021; Accepted 26 March 2021; Published 01 April 2021

Abstract

Considering a constant demand in construction of concrete structures to develop novel approaches for predicting the concrete’s properties, a host of investigations were performed on concrete’s mechanical properties and durability under various curing regimes. However, few studies were concerned with evaluating the concrete’s durability using non-destructive concrete surface resistivity tests by applying various curing conditions. The present study compares the influence of different curing regimes on durability and compressive strength of concrete to recommend the most effective curing conditions on concrete’s characteristics. Five curing conditions including ambient, laboratory, dry oven, wet oven and 7-days were analyzed. Accordingly, a non-destructive concrete surface resistivity test was performed on the concrete specimens using hand-held Wenner Resipod probe meter as a reliable and rapid approach. To analyze specimen’s durability, results of the surface sensitivity tests were correlated to chloride ion penetration rate based on the cylinder specimen dimensions and the degree of chloride ion penetration. The compressive strength tests were conducted on the specimens after 7, 28 and 56 days to determine the effect of curing conditions at different ages. Based on the reported outcomes, applying the wet oven curing regime results in higher compressive strength and durability compared to the other curing conditions.

Keywords: Concrete Curing; Compressive Strength; Durability; Electrical Resistivity.

1. Introduction

Various factors contribute to the degradation of the concrete specimens. Chloride ion penetration in concrete [1], weak materials [2], environmental conditions and curing conditions [3] are among these factors. There are several tools and techniques to measure the chloride penetration in concrete. The Rapid Chloride Penetration Test (RCPT) is the most popular chloride permeability measuring test [4]. Wenner Resipod probe meter as a novel approach provides non-destructive measurements which can be used to determine the surface sensitivity and chloride ion penetration [5]. The resistivity is determined by equation 1, as follows:

\[ \rho = \frac{2\pi LP}{I} \]  

(1)

Where \( \rho \) is the sensitivity \( \Omega \), \( L \) is the distance between two probes in meter, \( P \) is the potential in V and \( I \) is the applied current in the cross section [3]. Based on Florida Department of Transportation which is a premier in developing this method, surface resistivity of 12 or less should be considered as high chloride ion penetration, and consequently low concrete quality [6]. Electrical resistance could be used to control the durability of concrete specimens during
Concrete’s durability is governed by its penetration resistance to the harmful chemicals [9], and is affected by physical characteristics of concrete [10]. Using environmentally-friendly approaches such as incorporating fly ash in the concrete can reduces the harmful effect of concrete [11, 12]. Moreover, concrete members’s characteristics such as energy absorption can be improved by enhancing the durability and incorporating methods such as increasing the opening length in the layer concrete [13]. Concrete specimens manufactured with fine aggregate contribute to higher rate of chloride absorption and lower rate of durability [14]. Higher w/c ratio and porosity are the two other factors which could lead to lower durability in the final concrete product [15]. Improving the durability of concrete is achievable by adding the admixtures such as super plasticizers [16].

Curing condition is another factor with significant influence on concrete sample’s durability and strength [17], and permeability [18], especially in hot weather condition [19]. The effect of curing conditions on concrete’s performance, mechanical properties and durability of concrete specimens manufactured by recycled aggregate were investigated in some studies [20, 21]. In this regard, Thomas et al. [22] investigated effect of permeability on durability of recycled concrete specimens subjected to the corrosive condition. They exposed the specimens with various weights of the recycled aggregates along with different water/cement ratios to the marine environment. After curing the specimens in a humidity chamber, the durability of the aggregate concrete material reduced. Based on their research, cement paste’s quality affects the permeability of the specimens subjected to the aggressive environment. It was observed that the low water/cement ratios enhanced the specimens’ durability, considering the capillarity effect. Moreover, exposing to temperature is an influential parameter on concrete’s strength properties [23], and apparent resistivity [20]. Sabbağ and Uyanık [24] evaluated both unreinforced and reinforced concrete’s strength according to the apparent resistivity variations [25] and curing conditions. Both types of concrete samples were subjected to air and water cure. The potential differences on samples’ surfaces were measured using the electrical resistivity approach up to 90 days of curing in water saturated and dry conditions. They asserted that different curing condition affects concert’s strength and apparent resistivity. Based on their study, increasing the compressive strength of concrete reduces the resistive in the air cure; while it enhances it in the water cure. Appropriate curing leads to a significant improvement in concrete compressive and flexural strengths [26]. In addition, aggregate type and particle size are among other factors affecting the concrete properties such as the compressive strength [27, 28].

Curing is a necessary step to reach the acceptable compressive strength. It is also crucial to satisfy the minimum durability requirements. Curing conditions have significant influence on durability and strength of concrete specimens [29]. Various curing methods were investigated to enhance the concrete’s mechanical characteristics. For instance, the steam curing has been gained a wider attention to prepare precast structural elements with considerable strength at early ages. However, the steam curing adversely affects concrete’s properties by causing thermal damage on its microstructure. Therefore, application of other curing regimes on enhancing the concrete mechanical properties can be further investigated. In this context, Zou et al. [30] investigated the influence of subsequent curing on compressive strength and surface permeability of concrete subjected to steam curing. They measured surface permeability indexes regarding the air and permeation in concrete. Following the outcomes of their study, the subsequent curing regime significantly enhances concert’s compressive strength and permeability. Accordingly, the surface indexes were improved by 88.0 %. They reported the calcium hydroxide’s continuous dissolution as the main agent that can deteriorate the strength and impermeability of concrete during the steam curing. In another effort, Velandia et al. [31] performed a research study regarding the effect of curing, compressive strength and mix design on durability of concrete in the presence of fly ash. They asserted that the outdoor curing enhances permeability of concrete specimens. Moreover, they reported that the durability parameters are directly correlated with the compressive strength by using Portland cement in the concrete. Based on their observations, the microstructural agents affect the durability characteristics. Hassan et al. [32] casted Ultra-high performance concrete (UHPC) specimens reinforced with fibers to evaluate the effect of long-term curing in a 360-day period at 10, 20, 30, 90°C temperatures. They researched the effect of low temperatures (10-30°C) on tensile and compressive strength of the reinforced UHPC. Additionally, the outcomes were compared to the specimens subjected to the elevated temperate of 90°C. They concluded that the maximum strength for the curing at 90°C can be achieved at early ages (7 days). Also, specimens exposed to the lower temperatures reached to the acceptable compressive strength of 45 MPa in this period. Fallianno et al. [26] conducted a study to assess flexural and compressive strengths of lightweight concrete using different curing conditions. They considered three curing conditions (water, air, and cellophane) with different dry density and fiber contents. The reported negligible difference in the flexural strength of concrete specimens under different curing conditions. In another research, Hiremath et al. [33] investigated the influent of various curing conditions and duration on strength of reactive powered concrete at early ages. They asserted that the hot water bath curing regimes further enhances the concrete’s strength compared to standard water curing regime. Based on their results, the compressive strength of reactive powered concrete can be improved by almost 63% compared to the standard curing. Incorporating the most suitable curing condition is also beneficial in the precast concrete members.
1.1. Research Significance

Various studies have investigated the concrete’s mechanical characteristics and durability under different curing conditions. However, application of non-destructive concrete surface resistivity tests under various curing regimes to predict the concrete’s durability is not comprehensively studied. Furthermore, the present study. This research, compares the effect of different curing conditions on durability and compressive strength of concrete. The prime objective of this study is to recommend the most influential curing conditions. Accordingly, the required concrete properties in the construction process of concrete structures can be optimized by applying the effective curing conditions in the absence of additional harmful chemicals. In this context, an improvement on concrete’s mechanical characteristics without using additives can be deemed not only as a cost-effective method, but also an environmentally-friendly approach without incorporating harmful additive chemicals in the cement. Furthermore, the use of ordinary Portland cement, as the most common cement type, extends the applicability of the reported outcomes in this research. In addition, evaluating the effect of various curing regimes on mechanical properties of concrete results in improving the construction methods of pre-cast concrete members. In this regard, most suitable in-situ curing conditions can be applied to the pre-cased members by providing a controlled environment during the fabrication process. The present research study is devoted to evaluate the influence of various curing conditions on durability and mechanical characteristics of concrete using a hand-held Wenner Resipod probe meter. This electrical resistivity test as a reliable, non-destructive, and rapid method.

2. Materials and Methods

In the absence of any specified approach in the design codes on evaluation of durability behavior of concrete structures, characteristics of chloride diffusivity has been increasingly adopted as a performance-based approach to determine the durability of concrete structures, specifically in hazardous environment [34]. The concrete’s electrical resistivity can be characterized as the resistance versus the electrical current’s flow. Parameters such as the cement type, hydration degree, pozzolanic additives along with the water/cement ratio govern the concrete’s resistivity. Determination of concrete properties is achievable using the electrical resistivity test as a reliable, non-destructive, rapid method [35]. In addition, portable devices for measuring the resistivity can be used on either site or in-situ tests. Based on the developed specifications on the Wenner approach by ASTM, the relationship within the surface resistivity (Wenner four electrode approach) and the bulk resistivity (Wenner two electrode approach) is determined. Moreover, the relationship between the diffusivity and electrical resistivity is established by the Nernst–Einstein function [36], as follows:

\[
D_i = \frac{T \times R}{F^2 \times Z^2} \left( \frac{t_i}{\rho \gamma_i c_i} \right)
\]  

(2)

In this equation, the \(D_i\) stands for the ion’s diffusivity, \(R\), \(F\), \(Z\) are gas constant, Faraday constant, and ionic valence respectively, \(\gamma_i\) and \(t_i\) are respectively activity coefficient and ion’s transfer number. Also, \(\rho\) indicates the electrical resistivity and \(c_i\) shows the ion’s concentration (i) in the pore water. It is should be noted that for a specific curing condition a specific ion; other parameters such as faraday and gas constants as well as transfer number, activity coefficient, and ionic valence are assumed constant. Accordingly, the constant parameters are expressed using the \(K\) parameter to simplify the Equation 2 to the form below. For each of the three proposed mix designs, the ion concentration in the pore water remains constant [37],

\[
D = K \left( \frac{1}{\rho} \right)
\]  

(3)

Where \(K\) is the equivalent constant of the linear slope’s correlation within the electrical conductivity and diffusivity equal to the inverse amount of electrical resistivity. Considering the constant values of the mentioned parameters, the quality of electrical connection between the electrodes and concrete is mainly the function of distance between the electrodes [38] and geometry of the concrete specimens affects the test outcomes. Another important factor that governs the outcome of the resistivity test, is the environmental condition. Both the geometrical specification and environmental conditions (curing regimes) were considered in this study.

The electrical flow is carried through the pore solution by charged dissolved ions. In this regard, the electrical resistivity can be deemed as an index of concrete’s pore structure. The electrical resistivity rest was performed by a hand-held Wenner Resipod probe meter with 1K.

During this research investigation, three concrete mix were perpetrated to perform the experiments. Table 1 presents the properties and the combination of the materials used for each mix. Total of 135 specimens were prepared for this experiment which includes 45 for each mix design. All the prepared specimens were 4×8 cylinders, which were made according to the ASTM C172/C172M-17 specifications [39].

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Table 1. Mix properties

<table>
<thead>
<tr>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾ Rock</td>
<td>¾ Rock</td>
<td>¾ Rock</td>
</tr>
<tr>
<td>Type V – Cement</td>
<td>Type V – Cement</td>
<td>Type V – Cement</td>
</tr>
<tr>
<td>Apex Sand</td>
<td>Sand</td>
<td>Con Sand-3</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>Fly Ash</td>
<td>Fly Ash</td>
</tr>
<tr>
<td>Water Reducer WRDA 64</td>
<td>Water Reducer Type A and E</td>
<td>Con Sand-2 (crushed)</td>
</tr>
<tr>
<td>Retarder</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Water Cement Ratio (0.36)</td>
<td>Water Cement Ratio (0.46)</td>
<td>Water Cement Ratio (0.45)</td>
</tr>
</tbody>
</table>

In order to investigate the effects of curing conditions on concrete’s durability and compressive strength, the following five curing conditions were established:

a) Ambient condition: the samples were remained in ambient condition without any curing process;

b) 7-days curing condition: the samples were covered with a wet burlap and wet mat, in order to keep them in constant moist condition for a 7-day period;

c) Dry-oven condition: the specimens were kept in an oven and subjected to 120°F, during the performed experiment;

d) Wet-oven condition: the specimens were kept in a wet oven under 120 °F in the experiment period;

e) Laboratory standard condition: the specimens were placed in the standard moist room in the time of the experiment.

As presented in Figure 1, a step by step procedure was performed to collect the required data. The surface resistivity and compressive strength tests were conducted by maintaining the specimens in mentioned curing conditions for 7, 28 and 56 days. Figure 2 demonstrates resistivity test process. In order to perform this test, the surface of the concrete specimens was marked on the top circular finished surface, and then the resistivity test was conducted on each sample twice within approximately 5 minutes. After performing the resistivity test as a non-destructive experiment, the same samples were used for measuring the compressive strength test. In order to analyze the durability of the samples, the results of the surface sensitivity tests were correlated to chloride ion penetration rate based on the cylinder specimen dimensions and the degree of chloride ion penetration, in accordance with the AASHTO T358-19 specifications [40]. Sengul [41] reported the significant correlation of R=0.98 between the electrical conductivity and chloride diffusivity. The chloride diffusivity is a considerable factor for assessing the durability of concrete members. Various classifications are proposed for concrete’s chloride penetration resistance [42, 43]. Considering the relationship of electrical resistivity and diffusivity, the concrete’s resistivity can be applied to these classifications to indirectly evaluate the concrete’s resistance against the chloride penetration. During the measurement of electrical resistivity by the resistance meter, an ammeter, constant direct current using a power supply, the NaCL solution for filling up the cells and two voltage cells are required according to the ASTM specifications. Some issues such as proper mounting the concrete specimens within the two voltage cells along with potential of NaCL solution leakage can delay the estimated time for completing the tests and requires additional attention and effort for enhancing the reliability of test outcomes. Moreover, the variations of temperature and moisture due to different environmental conditions affect the outcomes of the measured resistance. Such different environmental conditions were investigated in this study using five curing conditions.

The correlation between surface resistivity and chloride penetration is presented in Table 2.

Table 2. Correlation between surface resistivity and chloride ion penetration

<table>
<thead>
<tr>
<th>Chloride ion penetration</th>
<th>Cylinder specimen (4” × 8”)</th>
<th>Cylinder specimen (6” × 12”)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>&lt;12</td>
<td>&lt;9.5</td>
</tr>
<tr>
<td>Moderate</td>
<td>12 - 21</td>
<td>9.5 – 16.5</td>
</tr>
<tr>
<td>Low</td>
<td>21 - 37</td>
<td>16.5 - 29</td>
</tr>
<tr>
<td>Very low</td>
<td>37 - 254</td>
<td>299 - 199</td>
</tr>
<tr>
<td>Negligible</td>
<td>&gt;254</td>
<td>&gt;199</td>
</tr>
</tbody>
</table>
3. Results

3.1. Surface Resistivity Test

The surface sensitivity tests were conducted in two ranges, high and low ranges using three specimens, and the average measured values were considered. Figures 3, 4 and 5 demonstrate the results of sensitivity tests performed on specimens made with the mix 1, after 7, 28 and 56 days, respectively.
Figure 3. Surface resistivity test results on mix 1 after 7-day curing

Figure 4. Surface resistivity test results on mix 1 after 28-day curing

Figure 5. Surface resistivity test results on mix 1 after 56-day curing
Comparing Figures 3 to 5 reveals that the lowest surface resistivity was reported for tests performed after 7 days. This is regardless of the curing condition. Although wet oven curing specimens show slightly higher resistivity after 7 days curing the chloride ion penetration for all specimens are considered high e.g. low durability. The 28-day test results indicate significant improvement in surface resistivity of wet-oven cured specimens followed by 7-days cured samples. The resistivity of specimens subjected to the laboratory and ambient curing conditions was slightly improved slightly but still be considered as “high” in terms of chloride ion penetration rate. The resistivity rate of samples maintained in the dry oven condition was not improved. After the 56-day curing time, the resistivity of specimens under laboratory curing condition has improved by 200 percent, however; it is not comparable to the results of wet oven curing condition. The chloride ion penetration rate for both laboratory curing and 7-days curing conditions are considered moderate, while for wet oven curing condition this rate is very low and therefore the durability of samples kept in wet oven curing condition for 56 days is considered to be very high. Dry oven and ambient curing conditions lead to very low resistivity. Figures 6, 7 and 8 illustrate the resistivity test results on the mix design 2 samples.

Concrete Mix Design 2, (7 days)

Figure 6. Surface resistivity test results on mix 2 after 7-day curing

Similar to the mix design 1, the test results after 7 days subjected to the different curing conditions indicate lower resistivity range compared to 28- and 56-days curing experiment periods. Wet oven and dry oven curing reveal higher resistivity compared to the standard, ambient and 7-day curing conditions. The durability for specimens under both wet and dry oven conditions are considered moderate. Specimens kept in wet oven curing condition present the highest surface resistivity after 28 and 56 days and the chloride penetration rate for both tests are considered very low. The specimens under 7-days curing conditions indicate better results for 28-days tests while the samples kept in standard laboratory condition show significant enhancement in resistivity rate after curing 56 days. Both dry oven and ambient curing conditions resulted in lower durability in comparison to the other curing conditions. Finally, Figures 9, 10 and 11 presents the performed test results regarding the mix design 3.

Concrete Mix Design 2, (28 days)

Figure 7. Surface resistivity test results on mix 2 after 28-day curing
Similar to the results presented for mix 1 and 2, the lower resistivity rates belong to the tests that performed on the samples being in curing condition for after 7 days. The reflected data on Figure 9 and Table 2 reveal such a resistively rate, with exception for the wet oven curing condition; which shows moderate penetration rate. The other four curing conditions resulted in very low resistivity condition which indicates a high penetration rate. Based on the results of mix 3 experiments after 28 and 56 days of curing time, are presented in Figures 10 and 11, standard laboratory curing and 7-days curing follow the patterns similar to mix 1 and mix 2. On the other hand, laboratory curing condition has considerably lower penetration rate after being kept in curing condition for 56 days.
3.2. Compressive strength Tests

In addition to the surface resistivity tests, the compressive strength tests were conducted on the specimens after 7, 28 and 56 days to determine the effect of various curing conditions at different ages on concrete samples.

As shown in Figure 12, the compressive strength values of the samples for mix design 1 at earliest curing period (7 days) is fairly comparable for samples under 7-day curing condition and laboratory condition. Similarly, dry oven and ambient curing regimes have close strength values at early ages. Accordingly, the minimum and maximum compressive strength values were measured 3310.64 and 457825 psi for specimens subjected to ambient and wet oven curing regimes, in the order given. Following the constructed tests after a 28-day period, the compressive strengths of specimens under 7-day and laboratory and wet oven conditions were improved by almost 25.14, 41.58, and 29.69%, compared to the 7-day experiment results, respectively. The maximum improvement between the 7 and 28 days was observed for the laboratory curing condition, which shows 1557.93 psi increase in the strength values. On the other hand, compressive strengths of specimens subjected to dry and ambient conditions is almost similar for the 7-day and 28-day test periods with the maximum improvement of 0.5 and 6.38% improvement, respectively. The similar trend of compressive strength enhancement was reported at the 56-day age, compared to the 7-day and 28-day ages. Based on the measurements on 56-day specimens, the maximum and minimum increases in the strength values were reported for
laboratory and dry oven conditions. This values show 25.62 and 3.66% enhancements compared to the 28-day specimens, respectively.

Curing conditions

Figure 12. Compressive strength test results on mix 1, subjected to various curing conditions

Figure 13 illustrates compressive strengths of concrete mix design 2 in 7, 28, 56 days for the concrete specimens subjected to the 5 different curing regimes. At the earliest age, the wet oven curing condition results in the maximum compressive strength of 6054.63 psi, while the minimum strength value was reported for the dry oven curing regime, 4583.8 psi. In this period, the strengths of specimens under all curing conditions except the wet oven are comparable. For instance, the difference of 568 psi is reported between the 7-day curing and the dry oven curing regimes. Outcomes of the compressive strength measurements of the 28-day specimens shows similar improvements in comparison with 7-day specimens. In this period, the highest compressive strength is recorded for the wet oven curing method which has an almost 35% of improvement compared to the 7-day period. In addition, the 7-day and laboratory curing conditions show more significant strength values compared to the dry oven and ambient curing regimes. The results of the compressive strength at the age of 56 days shows a similar trend to the 28-day specimens. At the 56-day age, wet oven and laboratory curing conditions has the most compressive strength values compared to the other curing methods. The lowest strengths in this period was recorded to the specimens cured by dry oven and ambient approaches.

Curing conditions

Figure 13. Compressive strength test results on mix 2, subjected to various curing conditions
Figure 14 presents the measured compressive strengths of the concrete specimens based on the mix design 3. In the 7-day curing period, the strength values of the specimen’s cured under laboratory, and dry oven regimes are approximately similar, with less than 50 psi difference. In this period, curing by wet oven shows the most promising results, while the minimum strength value belongs to the specimens cured under ambient condition. In the 28-day period, the maximum improvements were recorded for the specimens subjected to the laboratory and wet oven curing regimes with 48.5% and 36.22% enhancements of compared to the 7-day, respectively. It was observed that the compressive strengths of specimens with dry oven and ambient condition have improved between the 7-day and 56-day curing periods by merely 5.7% and 16.73%, respectively. The most significant compressive strength was reported for specimens under laboratory and wet oven curing conditions with over than 7000 psi of compressive strength.

### Concrete Mix Design 3

![Average Compressive Strength Chart](chart.png)

**Curing conditions**

Based on the presented measurements in Figures 12 to 14, the highest compressive strength was observed for specimens subjected to the wet oven condition, followed by laboratory curing condition for all the tested ages. In general, mix 1 specimens demonstrate lower compressive strength compared to the mix 2 and 3 for all curing conditions. Figures 12 and 14 illustrate that specimens kept in dry oven ambient curing condition did not meet the minimum required 4500 psi compressive strength, and are not acceptable. Moreover, it was observed that the aging has crucial impact on the compressive strength of the concrete. In this regard, almost all specimens present an improvement in compressive strength by increasing the curing period to 28 and 56 days.

### 4. Conclusions

In this study the influence of different curing conditions on the durability and compressive strength of concrete is investigated. Concrete specimens were prepared following the three different mix design types and were subjected to five curing conditions. In order to consider the effect of aging, the experiments were tested after 7, 28 and 56 days of being subjected to the curing process. The results of this investigation are summarized as follow:

- The presented test results show the importance of curing to achieve acceptable durability in the concrete samples;
- It was observed that the concrete specimens subjected to the wet oven curing condition has higher surface resistance compared to the other curing regimes. Correspondingly, for all the three mixes and at any testing age, the wet oven curing condition demonstrated significantly higher durability in comparison to standard laboratory curing condition;
- Specimens with wet oven curing has the most compressive strength values in the proposed mix designs compared to other implemented curing regimes. The laboratory curing procedure also demonstrate high compressive strength followed by 7-days curing condition;
Concrete specimens which were maintained in either of ambient or dry oven conditions did not show a considerable resistance against the chloride ion penetration. This specimen had lower durability compared to specimens with other curing methods;

Use of the dry oven and ambient curing regimes result in considerably low surface resistivity and compressive strengths. Accordingly, some specimens did not meet the minimum 4500 psi compressive strength failure criteria;

The specimens under 7-day curing did not show remarkable surface resistance or compressive strength, however by increasing the curing period, the strength and durability of this specimens were considerably improved. Such an improvement resulted in higher compressive strength of 7-day curing after 28 and 56 days compared to the specimens with dry oven and ambient conditions at the same ages;

Similarly, the surface resistivity of 7-day curing regime has considerably improved by aging and exceeds the resistance of dry oven and ambient regimes. For instance, in the mix design 1, the resistivity of concrete specimens under 7-day curing condition has improved by 184% and 220% at 28 and 56 days, compared to the earliest age.

5. Declarations

5.1. Author Contributions


5.2. Data Availability Statement

Data sharing is not applicable to this article.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Study of Head Loss in Rapid Filtration with four River Sands

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Received 16 January 2021; Revised 22 March 2021; Accepted 27 March 2021; Published 01 April 2021

Abstract

In this work, we studied the filtration behavior, with regard to the head loss, of four calibrated Togo Rivers sands compared to that of a reference filter sand imported from Europe. The objective is to determine the suitability of local rivers sands as filter sands for water treatment plants. The sands were successively loaded into a filtration pilot and subjected, during at least 20 hours, to the filtration of water whose turbidity was maintained at around 20 NTU. The results show that the average deviations of the head loss profiles as a function of depth, calculated in relation to the head loss recorded on the reference sand, at the same filtration time t=20h, are small and vary from 2 cm to 8 cm. In the same way, the curves of the head loss as a function of time are quite close to the one observed for the reference sand. Examination of the clogging front after 20 hours of filtration reveals that the progression is either the same or greater and reached 20 cm in depth at the same time. This study can be extended to other rivers sand samples and by varying the turbidity and the filtration rate.

Keywords: Filtration; Filter Sand; Head Loss; Clogging Front.

1. Introduction

In order to meet the many challenges related to universal access to drinking water, developing countries need to optimize the investment costs of drinking water production facilities. A better knowledge of local materials is one of the keys to taking advantage of existing potential at the territorial level. The efficient use of local materials requires a better knowledge of their characteristics and a thorough evaluation of their suitability for the intended uses. In Togo, as in most countries of the West African sub-region [1, 2], filter sand is generally imported for the needs of filter equipment in new drinking water treatment plants. In other way, if a drop of the level in the filtering beds is noted at the treatment plants, sandy materials are taken locally in the plant area to complete it, without knowing their qualifications as filtering material for a water intended for human consumption. A study of the physical and granular characteristics of Togolese river sands showed that these materials have good potential, in terms of normative values, to be used as a filter sand [3-5].

The present work aims to study the filtering capacities of Togolese four river sand samples that have been previously calibrated according to the characteristics of a reference filter sand, imported from Europe and used in a drinking water treatment plant in Togo.

From data on head losses resulting from experimentation under the same conditions, on a filtration pilot, the evolution of the head loss as a function of time and depth in the filtering beds made up of these materials was studied.

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The analysis of these data allowed a better understanding, with regard to the pressure drop, of the real filtration performances of the local materials studied, in comparison with the data recorded on the reference filter sand.

2. Experimental Approach

The river sands studied are taken from the sites of Adakpame (4ème lac), Dagué (lac Togo), Togocomé (lac Togo), Gogokondji (Mono). This choice was made to take account of the proximity of these sites to the water treatment plant of Lomé-Cacavelli, from which the reference sand of this study was taken. The location map of the maritime region of Togo with the river sands sampling sites is shown in Figure 1.

![Figure 1. Location map of the river sands sampling sites](image)

In fact, for comparison purposes, the same characteristics as those of the reference sand will be retained for the sands studied. The physical characteristics of the sands (Figure 5) are shown in the table 1 where D10 is the effective size, Cu the uniformity coefficient and P_U the usable proportion of the calibrated sand in the samples.

The schematic experiment set-up of the filtration is shown in Figure 3. The main components of the installation are described below. The dimensions (diameter, height...) of the pipes and the bed are chosen to simulate a rapid filtration under the conditions of the test. The components are:

- A cylindrical PVC pressure pipe PN 10, with an internal diameter of 90 mm and a total height of 150 cm;
- A 70 cm deep bed of uniform mono-media filter sand, constituting the filtering bed;
- Five tapping valves along the filtering bed to take raw water samples;
- A set of five piezometric tubes arranged along the bed at depths of 10, 20, 30, 40 and 55 cm for pressure drop measurements at different depths and equipped with a graduated ruler to read the water heights;
• An overflow made up for the evacuation of the excess water overlying the bed;
• A bottom tap to collect samples of filtered water;
• A peristaltic pump model YZ1515x from the manufacturer CRPUMP to supply the pilot with raw water;
• A raw water tank with a capacity of 150 litres.

Table 1. Physical characteristics of the sands [3, 6-9]

<table>
<thead>
<tr>
<th>Sampling sites</th>
<th>Adakpame (4ème lac)</th>
<th>Dagué (lac Togo)</th>
<th>Togocomé (lac Togo)</th>
<th>Gogokondji (Mono)</th>
<th>Cacaveli</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density</td>
<td>1.41</td>
<td>1.44</td>
<td>1.44</td>
<td>1.39</td>
<td>1.45</td>
</tr>
<tr>
<td>Absolute density</td>
<td>2.63</td>
<td>2.61</td>
<td>2.61</td>
<td>2.72</td>
<td>2.68</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>46%</td>
<td>45%</td>
<td>45%</td>
<td>49%</td>
<td>46%</td>
</tr>
<tr>
<td>Acid loss (%)</td>
<td>0.66</td>
<td>0.86</td>
<td>0.74</td>
<td>0.96</td>
<td>1.08</td>
</tr>
<tr>
<td>Friability (750 strokes %)</td>
<td>7.8</td>
<td>2.2</td>
<td>2.2</td>
<td>12.2</td>
<td>28.89</td>
</tr>
<tr>
<td>Friability (1500 strokes %)</td>
<td>12.22</td>
<td>4.44</td>
<td>4.44</td>
<td>22.22</td>
<td>46.67</td>
</tr>
</tbody>
</table>

Calibrating for $D_{10}=1.06$ mm and $C_{u}=1.53$

<table>
<thead>
<tr>
<th>P_{10} en %</th>
<th>10</th>
<th>10</th>
<th>8</th>
<th>22</th>
<th>-</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical and practical granular fractions (mm)</td>
<td>1 – 3.15</td>
<td>1 – 3.15</td>
<td>1 – 3.15</td>
<td>0.98 - 2</td>
<td>-</td>
</tr>
<tr>
<td>1 – 3.15</td>
<td>1 – 3.15</td>
<td>1 – 3.15</td>
<td>1 - 2</td>
<td>0 – 2.5</td>
<td></td>
</tr>
</tbody>
</table>

The five samples of filter sand were successively loaded into a filtration pilot and subjected, during an operation period of at least 20 hours, to the filtration of water whose turbidity was maintained at around 20 NTU in order to have comparable results for each sample studied. The filtration rate was adjusted, under laboratory conditions, to be close to the lower threshold for rapid filtration by placing it at around 2.74 m/hour [10] because to simulate rapid filtration, the filtration rate should be between 2 and 15 m/hour. During the operation period, a series of measurements were carried out on the pressure drop at the bottom of the filter and at different depths of the bed [11]. The analysis of the treatments results carried out on the basis of the data taken from the river sand samples will enable comparisons to be made with those of the reference sand in order to draw conclusions regarding the performance of the sands under study from the point of view of head loss.

The pressure drop measurement as a function of time gives an idea of the clogging evolution in the filter. In order to measure the pressure head at different depths of the filter bed, readings were taken on the piezometric tubes fitted with graduated rulers and fixed to the wall of the filter column. The variation of the pressure heights at different depths of the filtering medium during the filtration cycle was read from the water level in the corresponding piezometric tube. The total pressure drop across and along the filter bed at different depths was obtained from the difference in the corresponding pressure readings.

The data collected on the pressure drop allow to represent the evolution of the pressure drop as a function of time and as a function of the different depths of the filter bed. An assessment of the variation differences with the reference sand will also be made in order to draw conclusions on the behaviour of the materials studied, under the same filtration conditions.

Figure 2. Filtration experiment set-up
Figure 3. Schematic filtration experiment set-up
Specifically, the study of the pressure drop evolution as a function of time will allow to deduce, from the regression functions, the YVES [12] and DEGREMONT [13] models.

These two models of the pressure loss (h) as a function of time (t) are interesting for interpreting the pressure loss curves evolution encountered in practice. Above all, they make it possible to evaluate the time taken for a given filter to reach its maximum permissible pressure drop.

With the pressure loss data collected at different depths, the pressure gradient dp/dz is determined as a function of time for the different layers of the filter bed. The study of the pressure drop profiles evolution allows to follow the clogging of the bed. This diagram will thus allow a comparative analysis of the evolution of the clogging front inside the different beds, under the filtration conditions of our work.

3. Results and Discussion

The head loss variation, as a function of depth, based on interpolated data, for a filtration time of 20 hours, for the five samples, is as shown on the graph in Figure 7.
The analysis of this graph, carried out at the same filtration time $t$, for all the sand samples, confirms the uniformity of the variation of the pressure losses in the beds with slight deviations from the reference sand. Table 2 shows these deviations as a function of depth as well as the average deviation over 55 cm of filter bed.

It is noted that, generally, the average deviations of the head loss profiles as a function of depth, calculated in relation to the head loss recorded on the reference sand, at the same time, are small and vary from 2 cm for the Dagué sand to 8 cm for the 4th Lake (Adakpame) sand. We can deduct from this that the behavior, under the filtration conditions of our tests, of the river sands, after 20 hours of operation of the pilot, is close to that of the reference sand.

On the basis of the strong correlations observed for the pressure losses variation according to depth, the pressure losses at the bottom of the bed, at a depth of 70 cm, which could not be measured during the tests, were calculated from mathematical trend curve models, because the filtration pilot used did not have support gravel at its bottom to allow the installation of a piezometer at this depth. The curves of variations in pressure losses as a function of time for the 55 cm depths (data measured during the tests) and the bottom of the beds are shown in Figures 8 and 9 [14].

The curves confirm the existence of a linear relationship at the beginning of the filtration cycle (YVES model) [12], as can be seen here in the two figures between 0 and 5h and then the pressure drop increases exponentially (DEGREMONT model) [13]. The expressions of the linear (YVES) and exponential (DEGREMONT) type models, obtained from the data and curves in Figure 9 for total head losses in beds, are shown in Table 3 with the corresponding $R^2$ determination coefficients.

These expressions show, when analyzing the values of the determination coefficients, that the exponential type function best reflects the head loss variation as a function of time, under the adopted filtration conditions. In order to compare the behavior of the different filtering sands under study as a function of time, these models were used to calculate the head losses values at the times 0h, 5h, 10h, 15h and 20h for the different samples, as well as the deviations noted with the values of the reference filtering sand. The results are presented in Table 4.

### Table 2. Head loss gap ($\Delta h$) as a function of depths at time $t=20h$

<table>
<thead>
<tr>
<th>Bed depth (cm)</th>
<th>Cacaveli</th>
<th>Gogokondji</th>
<th>Togocome</th>
<th>Dagué</th>
<th>4ième Lac</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ah</td>
<td>Ah1</td>
<td>Ah2-Ah</td>
<td>Ah3</td>
<td>Ah4</td>
<td>Ah5-Ah</td>
</tr>
<tr>
<td>10</td>
<td>11.49</td>
<td>15.97</td>
<td>4.48</td>
<td>13.84</td>
<td>2.35</td>
</tr>
<tr>
<td>20</td>
<td>13.19</td>
<td>16.94</td>
<td>3.75</td>
<td>17.49</td>
<td>4.3</td>
</tr>
<tr>
<td>30</td>
<td>14.11</td>
<td>17.61</td>
<td>3.5</td>
<td>18.91</td>
<td>4.8</td>
</tr>
<tr>
<td>40</td>
<td>14.62</td>
<td>17.94</td>
<td>3.32</td>
<td>19.74</td>
<td>5.12</td>
</tr>
<tr>
<td>55</td>
<td>14.44</td>
<td>18.66</td>
<td>4.22</td>
<td>20.76</td>
<td>6.32</td>
</tr>
</tbody>
</table>

Average gap (cm) | 4 | 5 | 2 | 8

On the basis of the strong correlations observed for the pressure losses variation according to depth, the pressure losses at the bottom of the bed, at a depth of 70 cm, which could not be measured during the tests, were calculated from mathematical trend curve models, because the filtration pilot used did not have support gravel at its bottom to allow the installation of a piezometer at this depth. The curves of variations in pressure losses as a function of time for the 55 cm depths (data measured during the tests) and the bottom of the beds are shown in Figures 8 and 9 [14].

The curves confirm the existence of a linear relationship at the beginning of the filtration cycle (YVES model) [12], as can be seen here in the two figures between 0 and 5h and then the pressure drop increases exponentially (DEGREMONT model) [13]. The expressions of the linear (YVES) and exponential (DEGREMONT) type models, obtained from the data and curves in Figure 9 for total head losses in beds, are shown in Table 3 with the corresponding $R^2$ determination coefficients.

These expressions show, when analyzing the values of the determination coefficients, that the exponential type function best reflects the head loss variation as a function of time, under the adopted filtration conditions. In order to compare the behavior of the different filtering sands under study as a function of time, these models were used to calculate the head losses values at the times 0h, 5h, 10h, 15h and 20h for the different samples, as well as the deviations noted with the values of the reference filtering sand. The results are presented in Table 4.
Figure 8. Head loss variation as a function of time (55 cm depth)

Figure 9. Head loss variation as a function of time (70 cm depth)

Table 3. Expressions of total head loss as a function of time (filtration with \( V=2.74 \) m/h, 70 cm of bed depth and \( D_{10} = 1.06 \) mm)

<table>
<thead>
<tr>
<th>Sites</th>
<th>YVES Model</th>
<th>DEGREMONT Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \Delta h = f(t) )</td>
<td>( \Delta h = f(t) )</td>
</tr>
<tr>
<td></td>
<td>( R^2 )</td>
<td>( R^2 )</td>
</tr>
<tr>
<td>Cacaveli</td>
<td>( \Delta h = 0.4944t + 6.0854 )</td>
<td>( \Delta h = 6.7982e^{0.0417t} )</td>
</tr>
<tr>
<td>Gogokondji</td>
<td>( \Delta h = 0.7356t + 5.7932 )</td>
<td>( \Delta h = 7.3938e^{0.0403t} )</td>
</tr>
<tr>
<td>Togocomé</td>
<td>( \Delta h = 0.9468t + 5.2898 )</td>
<td>( \Delta h = 7.3377e^{0.0577t} )</td>
</tr>
<tr>
<td>Dagué</td>
<td>( \Delta h = 0.9915t + 3.777 )</td>
<td>( \Delta h = 6.3341e^{0.0821t} )</td>
</tr>
<tr>
<td>4ème Lac</td>
<td>( \Delta h = 1.2058t + 3.1307 )</td>
<td>( \Delta h = 5.955e^{0.0728t} )</td>
</tr>
<tr>
<td>Time (h)</td>
<td>Cacaveli</td>
<td>Gogokondji</td>
</tr>
<tr>
<td>---------</td>
<td>----------</td>
<td>------------</td>
</tr>
<tr>
<td>0</td>
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<tr>
<td>5</td>
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<td>9.42</td>
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<td>15.28</td>
</tr>
<tr>
<td>20</td>
<td>15.65</td>
<td>19.47</td>
</tr>
</tbody>
</table>

Average gap (cm) 1.94 3.37 2.17 8.51

Analysis of the data in Table 4, in relation to the graphs in Figures 8 and 9, reveals the following:

- The curves of head losses evolution versus time, for the Togolese river sand samples, are quite close to the one observed for the reference sand over an operating time of more than 20 hours, under the conditions of the pilot filtration test with an average difference ranging from 1.94 cm to 8.51 cm over the period; these differences have evolved with the evolution of the filtration test and the growth of the head losses over time;
- The filter sand obtained from the Gogokondji site on the Mono River shows the lowest average deviation, followed by samples from the Dagué and Togocomé sites on Lake Togo and finally that of the 4th lake in Adakpamé;
- The head loss values observed at the start of the pilot filtration are fairly close and between 5.96 cm and 7.39 cm (with 6.80 cm for the reference sand), with a mean of 6.76 cm and a standard deviation of around 0.63; the dispersion is greater towards the end of the operation;
- The curves obtained from the river sand samples studied are above that of the reference sand, except for the Dagué and 4th Lake samples for which lower values are recorded at start-up and during about 4 hours of filtration. This can be explained in part by the finer grain size of these samples which resulted in slightly higher pressure drops during the filtration.

Figure 10. Grain size distribution curves of the calibrated sands for filtration test

In fact, although the river sands studied were calibrated to have an effective diameter and a uniformity coefficient in line with the reference sand, the particle size compositions obtained after calibration are not strictly similar to those of the reference sand, as shown in Figures 5 and 10. In general, the sizing specifications of sands are represented in the form of a spindle within which several grading curves can be inscribed which comply with the target specification. Thus, it can be noted on the graph above that the proportion of fine elements, evaluated here by the passers-by with a 1.25 mm sieve, is higher for the river sands studied (between 31.49 and 42.3%) against 24.26% for the reference sand. The presence, in higher quantities, of these fine elements between the grains can create obstacles to the interstitial flow of water and cause the observed differences in head loss [15]. Finally, it should be noted that the reference sand, which is a commercial sand produced in an industrial environment with advanced technological processes for sieving, could present a better calibration than the river filtering sands studied, whose calibration was carried out under laboratory conditions;
• Generally, the small differences observed between the curves can be explained essentially by the relative variation in filtration during the tests (filtration rate, raw water turbidity and temperature) and the inhomogeneity of the filter beds, from one sample to another [16].

Lindquist diagrams show the distribution of material retained in the gravel pack as a function of depth [9, 17-20]. In order to be able to carry out an appropriate analysis, the data for the Lindquist diagram for the five samples at the same time t=20h was calculated by interpolating the test results. This diagram is shown in Figure 11.

Examination of this diagram also reveals important information on the comparative evolution of clogging fronts in filter beds. Indeed, it can be noted that after 20 hours of filtration, the clogging front progressions were already identical to those recorded at the end of the operation and presented above. It is the same for the reference and Gogokondji filter sand and varies from 0 cm on the surface of the bed to 10 cm in depth. For the samples from the other sites, the progression of the front was greater and reached 20 cm in depth at the same time. This conclusion supports the previous observation on the similarity, under the filtration conditions of our tests, of the capabilities of the Gogokondji filter sand with those of the reference filter sand.

4. Conclusion

The filtering capacities study of the samples, subjected to the pilot filtration test, showed that the river sands, calibrated according to the properties of the reference filter sand, have curves of head loss variation versus depth that follow the same function as the reference sand. The average deviations of the head loss profiles as a function of depth, calculated in relation to the head loss recorded on the reference sand, at the same filtration time t=20h, are small and vary from 2 cm for the Dagué sand to 8 cm for the 4th lake sand. In the same way, the curves of evolution of the head loss as a function of time, for the river sand samples under study, are quite close to the one observed for the reference sand over an operating time of more than 20h, with an average deviation ranging from 1.94 to 8.51 cm over the period. Examination of the clogging front after 20 hours of filtration reveals that the progression of the clogging front is the same for the reference filter sand and the Gogokondji filter sand, varying from 0 cm on the surface of the bed to 10 cm deep. For the samples from the other sites, the progression of the front was greater and reached 20 cm in depth at the same time.

In conclusion, it can be noted that, notwithstanding the small deviations observed on certain parameters related to the evolution of the head loss as a function of time and depth, the calibrated river sands of Gogokondji, Togocomé, Dagué and 4th Lake (Adakpame) showed, during the filtration test, a behaviour similar to that of the reference filter sand of Cacaveli, under the filtration conditions of our work. For a better knowledge of these materials, the study can be extended to other rivers sand samples and by varying the turbidity of the raw water, the bed depth and the filtration rate.

5. Declarations

5.1. Author Contributions

Conceptualization, K.E.A., Y.M.X.D.A., and I.P.; writing—original draft preparation, K.E.A., Y.M.X.D.A., and I.P.; writing—review and editing, K.E.A., Y.M.X.D.A., and I.P. All authors have read and agreed to the published version of the manuscript.
5.2. Data Availability Statement

The data presented in this study are available in article.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Complex Linkage between Watershed Attributes and Surface Water Quality: Gaining Insight via Path Analysis

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Received 05 January 2021; Revised 23 March 2021; Accepted 30 March 2021; Published 01 April 2021

Abstract

Understanding the influence of various variables on surface water quality is extremely important for protecting ecosystem health. The principal aim of this study is to assess the direct (DE), indirect (IE) and total effects (TE) of socio-economic, terrestrial and hydrological factors on surface water quality via path analysis through the lens of 15 sub-basins located on Indus basin, Pakistan. Four path models were selected based on Comparative Fit Index (CFI) = 0.999 value. First path model showed that rangelands having low population density decline river runoff which decreases instream Electrical Conductivity (EC) because of lower anthropogenic activities. Second path model depicted that croplands having higher population density enhance river runoff due to irrigation tail water discharge which decline instream EC because of dilution. Third path model showed that croplands with higher population density enhance river runoff which increases instream NO3 concentration because of unscientific application of irrigation water. Fourth path model unveiled that croplands enhance Gross Domestic Product (GDP) which enhance river runoff and instream NO3 concentration. To protect ecosystem health, Best Management Practices (BMPs), precision farming and modern irrigation techniques should be adopted to reduce irrigation tail water discharges containing pollutants entry in Indus River.

Keywords: Path Analysis; Water Quality; Socio-economic; Terrestrial; Hydrological.

1. Introduction

Catchment hydrology is mainly influenced by watershed topography which includes catchment area, shape, and slope [1–4]. Study on Xiangxi River revealed that 26% instream water quality variations was caused by topographic features [4]. Watershed slope has positive relationship with nutrients (Total Phosphorous (TP) and Total Nitrogen (TN)), suspended solid, Chemical Oxygen Demand (COD) and Biological Oxygen Demand (BOD) [5]. Elevation and mean slope have a positive linkage with dissolved oxygen (DO) while negative correlation with dissolved phosphorus, turbidity, EC, COD, water temperature, TN, TP and NO3-N [4, 6]. Some studies reported that standard deviation of
slope enhance the contaminant concentration in surface waters [7, 8]. Literature shows that elevation and mean slope speed up soil erosion which pollute surface waters [4, 9].

Dynamics of land use, socioeconomic activities and fertilizer application causes spatiotemporal variation in surface water quality [10]. Fertilization and irrigation have strong consequences on water-land use relationship [11]. Under and over fertilization have strong effects on surface water quality and soil nutrition [35]. Water quality deterioration (nutrients and fecal indicator bacteria) is linked with animal grazing which threatens human health and the environment [12]. Population density, animal farming and dissolved phosphorous has high influence on GDP, EC, TP, and turbidity [3, 13]. Total suspended solids, NO₃+NO₂, Cu, Zn, oil and grease have strong linkage with human population density [3, 14, 15]. Intense anthropogenic activities and fertilizer applications for high crop yield badly impacts river water quality [16]. Therefore, it will be significant to reveal the possible complex terrestrial, socioeconomic and hydrologic impacts on surface water quality.

Surficial geology in addition to anthropogenic biomes is primary factors which should be considered in land management policies formulation [17]. Land use-surface water quality relationship has been extensively evaluated by researchers [18, 19, 20]. Intense agricultural activities and urban sprawl has severely affected the aquatic ecosystem health i.e., high overland flow, considerable trace elements and nutrients loads [3, 21]. These relationships suggest the unavailability of safe water for human consumption in near future [4, 9].

Precipitation and irrigation tail water discharges fuel the problem of water quality impairment from nonpoint sources of croplands [19]. NH₄-F and NOX-F have strong correlation with cropland biomes [22]. Literature shows that agriculture lands enhance nutrients concentration in surface water bodies which is mainly attributed to irrigation tail water discharges and precipitation overland flow which sweeps all kinds of pollutants especially fertilizer remaining’s from the top fertile soil layer to nearby water bodies [5, 23].

Rangelands play key role in controlling nonpoint source pollution. Dissolved organic carbon and NOX-F are negatively associated with rangeland biomes [24]. Rangeland’s biomes have lower human interference as well as good nutrients retention capacity which help in water environment protection [25-27]. Degradation of rangeland leads to degradation of water environment [28, 29].

Literature shows that researchers linked socioeconomic, topographic, terrestrial, and hydrological determinants with surface water quality separately [30]. The novelty of the current study is that it simultaneously assesses the DE, IE and TE of the aforementioned determinants on surface water quality. The major focus of this study is to evaluate the complex DE, IE and TE of terrestrial, socioeconomic and hydrologic variables on surface water quality in Indus basin using path analysis approach. Generally, path analysis is used to assess anthropogenic DE and IE on instream water quality. Furthermore, the notion of the current hypothetical model is completely based on literature that how does watershed socioeconomic, terrestrial and stream characteristics interact with each other?

2. Study Area, Data Collection and Methods

2.1. Study Area

Indus river basin is stretched over four countries with total covered area of 1.12 million Km² in which Pakistan, India, China, and Afghanistan covers 47, 39, 8 and 6% area. Indus river basin spreads over 520000 km² which covers 65 % territory of Pakistan. Annual precipitation varies geographically from 100 to 500 mm in low elevated areas and 2000 mm in high elevated areas. The flow regime is mainly governed by snowfall at higher altitudes. Indus basin climate varies greatly.

The present study is based on Indus river basin, Pakistan. This study covers fifteen water quality monitoring sites which include Barasin, Draband, Bisham Qila, Bunji, Dadu Moro Bridge, Gunji Bridge, Kachura, Khairabad, Kharmong, Mandori, Massan, Partab Bridge, Rikot, Shatial Bridge, and Sehwan as demonstrated by Figure 1.
2.2. Data Collection

2.2.1. Water Quality and Hydrological Data

Both discharge and water quality data were obtained from Water and Power Department (WAPDA) Pakistan for a period of (1963-2009). The current study covers fifteen monitoring stations which include Barasin, Draband, Bisham Qila, Bunji, Dadu Moro Bridge, Gunji Bridge, Kachura, Khairabad, Kharmong, Mandori, Massan, Partab Bridge, Rikot, Shatial Bridge, and Sehwan. This study is based on twenty-fundamental water quality variables which includes Cl, HCO$_3$, Ca, Mg, Na, K, SO$_4$, NO$_3$, CO$_3$, F, Total Cations and Anions, SiO$_2$, Fe, B, D.S by Evaporation, EC ×10$^6$ at 25°C, pH, Residual Carbonate me/l, and Sodium Adsorption Ratio (SAR)).

2.2.2. Socioeconomic Data

Digital Elevation Model (DEM) data, downloaded from Shuttle Radar Topography Mission (SRTM), was used for delineating sub-watersheds. For the delineated sub-watersheds, population density data was extracted from Socioeconomic Data and Applications Center (SEDAC) [31]. GDP data was obtained from the provincial economies report [32]. Moreover, provincial level GDP data were assigned to the delineated sub-watersheds.

2.2.3. Terrestrial Determinants Data

Anthropogenic biomes are classified in six main groups which are demonstrated by (Figure 2). Socioeconomic Data and Applications Center (SEDAC) source was used for downloading anthropogenic biomes data. In addition, surficial geology data that includes silt, sand, clay, and gravel was collected from the World Harmonized Soil Database (HWSD).
2.3. Methods

2.3.1. Watershed Modeling

The Digital Elevation Model (DEM) was utilized for delineating sub-watersheds using monitoring points as outlet via spatial analysis tool of Geographical Information System (GIS). DEM data was extracted for the study area as obvious from the (Figure 3). Land use, soil and population data was extracted using GIS. The extracted variables were linked with water quality parameters using path analysis technique. Flowchart demonstrating the methodology of the research is obvious from Figure 5.
2.3.2. Statistical Modeling

Structural equation modeling (SEM) is a multivariate statistical framework which was used to compute direct and indirect complex linkages among social variables, terrestrial variables, river discharge and surface water quality parameters. A hypothetical model of interrelationships among social variables, terrestrial variables, hydrological and water quality parameters is demonstrated by Figure 4.

![Figure 4. Hypothesized direct and indirect linkage among social factors, terrestrial factors, river discharge and surface water quality parameters. Arrows show associations among hypothetical variables.](image)

**Figure 4. Hypothesized direct and indirect linkage among social factors, terrestrial factors, river discharge and surface water quality parameters. Arrows show associations among hypothetical variables.**

2.3.3. Data Analysis

The current study is based on mean values of each variable of the conceptual hypothesized path model. Path analysis was carried out to model relationships (DE and IE) among social, terrestrial, hydrological and water quality parameters. The hypothesized model based on four attributes groups was selected as per the available literature. Variables with the probability of strong relationship with certain variables within each of the four groups were also selected based on the previous studies. Path analysis was carried out on a variety of hypothetical models to examine the relationship among various combinations of attributes variables and to choose small group of models that best explains linkage between the social, terrestrial, hydrological and water quality variables. Path coefficients (standardized) were derived from path analysis by simultaneously regressing a dependent variable on each independent variable immediately linked to it by an arrow in the hypothetical path model. Path coefficients were used to evaluate a dependent variable's effects (DE, IE and TE). Independent-dependent variable DE is equal to the corresponding path coefficient. Independent variable's IE was calculated by multiplying coefficients in every path from the independent variable across all of the mediating variables to the dependent variable, and then adding values for all indirect paths [33]. An independent variable's cumulative influence on a dependent variable is equivalent to the number of all DE and IE. The path analyses were performed using AMOS Version 19.0, a Structural Equation
Modeling (SEM) and path analysis program. The performance of various path models was assessed using Comparative Fit Index (CFI). CFI is preferred for small sample size studies. The CFI value ranges from 0 to 1. Path model having CFI > 0.9 is considered good fit [34].

3. Results and Discussion

Statistical assessment of path models showed that all the four path models are characterized by acceptable CFI values. All the four path models have CFI = 0.999 which shows that modeled values are in close agreement with observed values.

3.1. Path Model: EC as Output Variable

Mean population and discharge are the significant determinants which cause variations in stream EC as obvious from (Figure 6). More specifically mean population has strong direct relationship with riverine flow regime. Pollutants including sediments and heavy metals (Zn, Cu, Pb, etc.) that enhance EC may be discharged to Indus River due to anthropogenic activities i.e. surface runoff, ground water discharge, industrial effluents, soil erosion etc. [35-38]. Mean population has negative linkage with rangelands because rangelands are thinly populated areas. Rangelands are negatively associated with riverine flow regime which may be due to high infiltration owing to its pervious surface area [22, 25-27]. Basin having higher ration of rangelands decline pollutants load in surface waters owing to lower elasticity value and higher pollutants retention capability. Conservation of rangelands at watershed scale can help to protect ecosystem health. Riverine flow regime has negative correlation with EC which might be due to dilution effects [22]. Moreover, 2/3 climate models demonstrated that annual average runoff would increase in future [17]. In conclusion the mean population has strongest direct increasing while riverine flow has direct decreasing effect on EC. The total effect of mean discharge on EC is greater than the mean population and rangelands as obvious from Table 1.

<table>
<thead>
<tr>
<th>Dependent variable</th>
<th>Mean population</th>
<th>Rangeland</th>
<th>Mean discharge</th>
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<tbody>
<tr>
<td></td>
<td>DE</td>
<td>IE</td>
<td>TE</td>
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<td>Rangeland</td>
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<tr>
<td>EC</td>
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<td>-0.511</td>
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</table>

Figure 6. Social, terrestrial, and hydrological variables DE and IE influence on surface water EC

Mean population and riverine flow are the significant determinants which cause variations in stream EC as obvious from (Figure 7). More specifically mean population has strong direct relationship with riverine flow regime. Pollutants including sediments and heavy metals (Zn, Cu, Pb, etc.) that enhance EC may be discharged to Indus River due to anthropogenic activities i.e. surface runoff, ground water discharge, industrial effluents, soil erosion etc. [35-39]. Mean population has positive linkage with croplands because Indus is the largest basin in Pakistan where people live and grow crops. Riverine flow regime has negative linkage with EC which might be due to dilution effects [24]. Moreover, 2/3 climate models demonstrated that annual average runoff would increase in future [17]. Croplands are negatively associated with instream EC which may be due to irrigation tail water discharges which enhance riverine flow, causing dilution effect, owing to unscientific application of irrigation water in the Indus basin [40-42]. In conclusion the mean population has strongest increasing while riverine flow has decreasing effect on EC. The total effect of mean discharge on EC is greater than the mean population and croplands as obvious from (Table 2).
Wu et al (2015) found that path model having EC as outcome variable demonstrated that road density, population density and base flow ratio causes variation instream EC. The impacts of human population density on EC are positively and negatively mediated by road density and base flow ratio respectively. Results of the current study are supported by the aforementioned research work [14].

Table 2. Mean population, cropland, and mean discharge DE, IE and TE influence on instream surface water EC

<table>
<thead>
<tr>
<th>Dependent variable</th>
<th>Mean population</th>
<th>Cropland</th>
<th>Mean discharge</th>
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<td>IE</td>
<td>TE</td>
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</tr>
<tr>
<td>Mean discharge</td>
<td>0.759</td>
<td>0.759</td>
<td></td>
</tr>
<tr>
<td>EC</td>
<td>1.036</td>
<td>-0.578</td>
<td>0.459</td>
</tr>
</tbody>
</table>

Figure 7. Social, terrestrial, and hydrological variables DE and IE influence on surface water conductivity

3.2. Path Model: Total Nitrogen as Outcome Variable

Mean population and river runoff are the significant determinants which cause variations in NO\textsubscript{3} concentration as obvious from (Figure 8). River runoff has the strongest direct impact on riverine NO\textsubscript{3} concentration. Mean population has positive linkage with croplands because Indus is the largest basin in Pakistan where people live and grow crops. Riverine flow regime has positive linkage with NO\textsubscript{3} because nitrogen flux substantially increases with precipitation [24]. Pakistan lies in the temperate climate zone, generally arid climate, characterized by cold Winters and hot Summer, where the sensitivity of riverine flow to precipitation and snowmelt is higher which fuels the top soil erosion which enhances riverine NO\textsubscript{3} concentration. Similarly mean population has increasing effect on riverine NO\textsubscript{3} which may be due to high anthropogenic activities in Indus basin. Areas with high population density intensify nitrogen loads in riverine waters [5, 23, 28, 43-48] which may be due to intense human activities and low retention capacity in such areas [49]. Intense storms increase overland flow which sweeps domestic sewage and nutrients to nearby water bodies [19]. Croplands are negatively associated with instream NO\textsubscript{3} concentration which may be due to irrigation tail water discharges which enhance riverine flow, causing dilution effect, owing to unscientific application of irrigation water in the Indus basin [40-42]. The total effects of riverine flow regime on NO\textsubscript{3} concentration overweighs mean population in Indus basin as obvious from (Table 3). This study indicates that streams in this landscape will obtain more contaminants from various human activities [40-42]. It is utmost important to focus on the possible adverse consequences of high nitrogen rates. Various BMPs such as vegetated buffer strips [50, 51] or nitrogen reducing bioreactors should be adopted in such conditions [51, 52].

Table 3. Mean population, cropland, and mean discharge DE, IE and TE on instream NO\textsubscript{3}

<table>
<thead>
<tr>
<th>Dependent variable</th>
<th>Mean population</th>
<th>Cropland</th>
<th>Mean discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DE</td>
<td>IE</td>
<td>TE</td>
</tr>
<tr>
<td>Cropland</td>
<td>0.370</td>
<td>0.370</td>
<td></td>
</tr>
<tr>
<td>Mean discharge</td>
<td>0.759</td>
<td>0.759</td>
<td></td>
</tr>
<tr>
<td>NO\textsubscript{3}</td>
<td>0.030</td>
<td>0.223</td>
<td>0.252</td>
</tr>
</tbody>
</table>
GDP and river runoff are the significant determinants which cause variations in NO$_3$ concentration as obvious from (Figure 9). But river runoff has the strongest direct impact on riverine NO$_3$ concentration. GDP has positive linkage with croplands because Pakistan is an agricultural country where GDP mainly depend on agricultural products and Indus is the largest basin in Pakistan where people grow crops. Croplands are positively associated with riverine flow regime and negatively linked with NO$_3$ concentration. The aforementioned association may be due to irrigation tail water discharges which enhance riverine flow, causing dilution effect, owing to unscientific application of irrigation water in the Indus basin [40-42]. Riverine flow regime has positive linkage with NO$_3$ concentration because sensitivity of riverine flow to precipitation is higher in Pakistan which enhances nitrogen flux. Surface runoff sweeps and erodes the top fertile soil layer which increase riverine NO$_3$ concentration [24]. GDP is positively linked with riverine NO$_3$ concentration which may be due to urbanization and agriculture activities in Indus basin. Economic growth enhances urban development and agriculture activities where domestic sewage and irrigation tail water discharges enhance riverine NO$_3$ concentration [22]. This study indicates that streams in this landscape will obtain more contaminants from anthropogenic activities [40-42]. It is utmost important to focus on the possible adverse consequences of high nitrogen rates. Various BMPs such as vegetated buffer strips [50, 51] or nitrogen reducing bioreactors should be adopted in such conditions [51, 52].

Wu et al (2015) found that Path model having total nitrogen as outcome variable demonstrated that population density, riverine flow, % crop land causes variation in stream nitrogen concentration. The influence of human population density on total nitrogen is positively mediated by riverine flow [14]. The findings of the aforementioned research work support our study results.

<table>
<thead>
<tr>
<th>Dependent variable</th>
<th>GDP</th>
<th>Cropland</th>
<th>Mean discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DE</td>
<td>IE</td>
<td>TE</td>
</tr>
<tr>
<td>Cropland</td>
<td>0.218</td>
<td></td>
<td>0.218</td>
</tr>
<tr>
<td>Mean discharge</td>
<td>0.264</td>
<td>0.049</td>
<td>0.313</td>
</tr>
<tr>
<td>NO$_3$</td>
<td>0.23</td>
<td>0.072</td>
<td>0.302</td>
</tr>
</tbody>
</table>

Figure 8. Social, terrestrial, and hydrological variables DE and IE on surface water NO$_3$
4. Conclusion

Indus basin has complex socioeconomic, terrestrial, and hydrological conditions for which it can be difficult to identify all possible relationships and mechanisms which deteriorate surface water quality. Here we assessed the DE, IE and TE of socioeconomic, terrestrial and hydrological variables on stream water quality. Instream water EC increases with croplands while decreases with rangelands. The results demonstrated that watersheds having rangeland biomes pose lower risk to instream water EC impairment as compared to cropland. Similarly, instream water NO₃ concentration increases with mean population, GDP, and river flow. The results implied that in stream NO₃ concentration is enhanced by anthropogenic activities and river flow. Goodness of fit statistics demonstrated that path models are stronger. Small number of parameters entered the models because of small sample size (15 monitoring sites). The variables were selected based on literature to quantify the principal cause and effect models. Difference in linkage of socioeconomic, terrestrial, and hydrological variables with water quality parameters indicated that different activities impact certain water quality parameter differently which should be addressed while formulating water policies. To control elevated level of NO₃ concentration BMPs such as vegetative buffer strips, nitrogen reducing bioreactors should be adopted. Overall, the findings of the current study indicated that multiple adaptation strategies should be adopted to protect stream health.

5. Declarations

5.1. Author Contributions

Conceptualization, A.U.K.; data collection and processing, H.R., M.I.K., H.M.K. and A.U.K.; analysis and interpretation of the data, A.U.K.; writing original draft preparation, F.A.K. and A.U.K.; writing, review and editing, K.H., L.A., L.A.S., J.K., I.A. and A.A. All authors have read and agreed to the published version of the research article.

5.2. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


[43] Galbraith, Lis.. "Buffer Zone Versus Whole Catchment Approac


Performance Evaluation for Mechanical Behaviour of Concrete Incorporating Recycled Plastic Bottle Fibers as Locally Available Materials

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1 Hajee Mohammad Danesh Science and Technology University, Basherhat, Dhaka - Dinajpur Highway, Dinajpur 5200, Bangladesh.

Received 10 January 2021; Revised 16 March 2021; Accepted 22 March 2021; Published 01 April 2021

Abstract

The objective of the study is to investigate the influence of Polyethylene Terephthalate (PET) recycled plastic bottle fibers on the compressive strength and cracking of concrete. In this study, two types of fiber are used: straight and zigzag fibers whose length and aspect ratio are 40 mm and 40 respectively. 0, 0.75, and 1.25% volume fractions of fibers replacing the volume of coarse aggregates are used in this investigation. According to ACI 211.1-91, design mixing ratio 1:2:3 for M20 concrete and water-cement ratio 0.58 are used. Curing is done in field condition and weathering action is allowed in curing time. The destructive compressive strength test shows that the compressive strength of plain concrete is 19.84 MPa, at 0.75 and 1.25% replacement for concrete with straight fibers are 19.54 and 18.84 MPa, and at 0.75 and 1.25% replacement for concrete with zigzag fibers are 18.49 and 15.69 MPa. The non-destructive compressive strength test shows that the compressive strength of plain concrete is 13.58 MPa, at 0.75 and 1.25% replacement for concrete with straight fibers are 10.36 and 8.82 MPa, and at 0.75 and 1.25% replacement for concrete with zigzag fibers are 8.21 and 8.10 MPa. The use of fibers changes the failure mode. The addition of fibers decreases the workability and cracking of concrete. Zigzag fiber slightly shows interlocking property with concrete. The addition of PET plastic fibers increases the ductility of concrete.

Keywords: Compression Strength; Failure Mode; PET Fiber; Recycled Plastic Bottle Fiber; Straight Plastic Fiber; Zigzag Plastic Fiber.

1. Introduction

Due to the rapid growth of industrialization & urbanization around the world, lots of infrastructure developments are taking place and environmental pollution is occurring due to the use of non-biodegradable products. This present condition has led to the lacking of construction materials, including raw materials of concrete, leaving environmental pollution concerns. Coarse aggregates constitute the largest portion of concrete mixtures. Therefore, a more satisfactory replacement to natural coarse aggregate is necessary.

According to a study conducted by Waste Concern, a Bangladeshi social business enterprise that promotes resource recovery from waste, concluded that approximately 0.8 million tons of plastic waste is generated per year in Bangladesh of which 36% is recycled while 39% is landfilled and the rest 25% goes unchecked and finds its way into the marine environment [1]. So, plastics should be disposed of in a systematic way to save our environment. Plastic
waste has several characteristics, including being very difficult to decompose naturally. Plastic is a non-biodegradable product that is harmful to our environment. Plastics take about 500 years for decomposing. Reusing plastic waste in concrete is an effective way to reduce plastic pollution.

The Society of the Plastics Industry (SPI) made a detailed classification of plastic materials for plastic users and recyclers. An SPI code or number is molded into the bottom of the plastic so that the user can identify their desired material. The plastic materials are classified into seven types, namely Polyethylene terephthalate (PETE or PET), High-density Polyethylene (HDPE), Polyvinyl Chloride (PVC), Low-density Polyethylene (LDPE), Polypropylene (PP), Polystyrene and other plastics (https://www.scrantonproducts.com/different-types-of-plastics-and-the-spi-codes/). Polyethylene Terephthalate, known commonly as PET or PETE is best known as the clear plastic used for water, soda bottles containing and domestic purpose, etc. As a raw material, PET is globally recognized as a safe, non-toxic, strong, lightweight, flexible material that is 100% recyclable.

Reusing waste plastic as a sand-substitution aggregate in concrete gives a good approach to reduce the cost of materials and solve some of the solid waste problems posed by plastics [2]. For mechanical property enhancement, fiber reinforcement has been employed in various concrete structures to increase impact resistance [3], and particularly lighter weight concrete which could reach 68.88% lighter than concrete with virgin aggregates [4]. Foti has suggested a possible use of PET fibers in the form of flat or round bars, or networks for structural reinforcement [5]. Foti has also reported the possibility of using fibers from polyethylene terephthalate (PET) bottles to increase the ductility of the concrete [6]. Kim et al. have investigated that compressive strength and elastic modulus both decreased as PET fiber volume fraction increased. Cracking due to drying shrinkage was delayed in the PET fiber reinforced concrete specimens, compared to such cracking in non-reinforced specimens without fiber reinforcement, which indicates crack controlling and bridging characteristics of the recycled PET fibers [7]. Basha et al. have developed a correlation model between the mixture parameters, mechanical and thermal properties of concrete incorporating recycled plastic aggregate. The study revealed the potential of reducing thermal conductivity to 35–65% and give an opportunity of using the product as thermal insulation [8]. Saikia and de Brito have evaluated the effects of size and shape of recycled polyethylene terephthalate (PET) aggregate on the fresh and hardened properties of concrete. The results indicate that the slump of fresh concrete increases slightly with the incorporation of pellet-shaped PET-aggregate. Flakier plastic aggregate sharply decreases the slump of the fresh concrete and it further decreases if the content and size of this type of PET-aggregate increases. The compressive strength, tensile splitting strength, modulus of elasticity, and flexural strength of concrete deteriorate due to the incorporation of PET-aggregate and the deterioration of these properties intensifies with the increasing content of this aggregate [9]. However, there are some disadvantages in the application of PET fiber. Some of the disadvantages of the use of plastic fiber are lower alkali resistance [10], lower flexural strength [11], and lower dynamic elastic modulus [12].

It is well known that concrete is strong in compression and has high brittle property. But concrete is weak in ductility and toughness. With the addition of fibers and allowing little variation in compressive strength from plain concrete, these disadvantages can be overcome. It is also known that deformed rebar is used for reinforcing the concrete because deformed rebar gives interlocking property and gives a good bonding with concrete. In this study, straight fibers where no deformed shape is given and deformed shape zigzag pattern type fibers are used with concrete to study the mechanical properties of concrete.

This study describes the properties of workability (slump test, bleeding, and segregation), destructive and non-destructive compressive strength using recycled plastic bottle fibers as a percentage replacement with coarse aggregate. Recycled plastic bottle fibers are prepared in two forms, straight and zigzag fibers. A plastic cutter machine is used to prepare straight fibers and a zigzag pattern-making device is used to give the fibers a zigzag pattern. This research differentiates the behavior of straight and zigzag fibers in fresh and hardened concrete. The paper is categorized into four parts, where the introduction, covering some background and previous research on the topic. The next part presents the methodology of the experimental research, consisting of the preparation of materials, concrete mix design, test specimens, and test procedures. The next part presents the result and discussion of the fresh and hardened properties of concrete. The conclusion is presented at the end of the paper.

2. Methodology

Materials required for ten sets of concrete specimens were collected. Various properties of materials were tested and using these properties, the designing mixing ratio was calculated according to ACI 211.1-91 [13] for M20 concrete. The specimens were cast and curing was exposed to the environment. After that destructive and non-destructive compressive strength tests were done. All the specimens were tested after 28 days curing period. The research consists of several stages; material preparation, mix design, manufacture of test specimens, maintenance, and testing. Figure 1 shows the experimental procedure of the research.
2.1. Materials

Portland Composite Cement (PCC), locally available fine sand, and crushed stones were used for casting concrete specimens. Recycled plastic bottles were collected for making plastic fibers.

2.1.1. Coarse Aggregate

Crushed stones were used as coarse aggregate. The maximum size of aggregate was 20 mm and gradation of coarse aggregate was done conforming to ASTM C33 [14]. Specific gravity, unit weight, absorption capacity, and fineness modulus were found 2.584, 1539.362 kg/m$^3$, 0.838%, and 6.96 respectively.

2.1.2. Fine Aggregate

Locally available sand was used as fine aggregate. Specific gravity, unit weight, absorption capacity, and fineness modulus were 2.136, 1602.373 kg/m$^3$, 6.383%, and 2.756 respectively. Gradation of fine aggregate was done conforming to ASTM C33 [14].

2.1.3. Binder

The binding material was Portland Composite Cement (PCC) which contains 65%-79% clinker, 0-5% gypsum, slag, fly ash, and limestone 21%-35%. Specific gravity, initial setting time, final setting time, and normal consistency were 3.15, 125 min, 210 min, and 28.5% respectively.
2.1.4. Recycled Plastic Fibers

Fibers having length 40 mm, width 4 mm, thickness 0.2 mm, and equivalent diameter 1 mm were used both for straight and zigzag fibers. The aspect ratio (Length/Diameter) was 40. The unit weight of fiber was 1370 kg/m$^3$. A plastic bottle cutter and zigzag pattern-making device were used to give the fibers proper shape. Figure 2 shows two types of fiber. The equivalent diameter of plastic fiber and aspect ratio was determined by Equations 1 and 2 respectively.

\[
T \times B = \frac{\pi}{4} \times D^2 \tag{1}
\]

\[
\text{Aspect ratio} = \frac{L}{D} \tag{2}
\]

Where, \(L\) = Length of fiber; \(T\) = Thickness of fiber; \(B\) = Width of fiber; \(D\) = Equivalent diameter of fiber.

![Figure 2. (a) Straight plastic fibers; (b) Zigzag plastic fibers](image)

2.2. Concrete Mix Design

The mix was designed as per ACI 211.1-91 for M20 grade concrete with a 0.58 water-cement ratio. Concrete mixes were prepared by 0%, 0.75%, and 1.25% volume replacement of coarse aggregate with straight and zigzag fibers respectively. The percentage of the volume of coarse aggregate was taken and it was multiplied with the unit weight of fiber and thus the weight of fibers was taken. Then the weight of coarse aggregate was replaced by the weight of fibers. Table 1 shows the mixing proportion of concrete.

<table>
<thead>
<tr>
<th>S.N.</th>
<th>% of CA replacement</th>
<th>Cement (kg)</th>
<th>Sand (kg)</th>
<th>Stone (kg)</th>
<th>Water (kg)</th>
<th>Fiber (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0% PF</td>
<td>360</td>
<td>801.19</td>
<td>1154.52</td>
<td>145</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.75% ST PF</td>
<td>360</td>
<td>801.19</td>
<td>1146.81</td>
<td>145</td>
<td>7.71</td>
</tr>
<tr>
<td>3</td>
<td>0.75% ZZ PF</td>
<td>360</td>
<td>801.19</td>
<td>1146.81</td>
<td>145</td>
<td>7.71</td>
</tr>
<tr>
<td>4</td>
<td>1.25% ST PF</td>
<td>360</td>
<td>801.19</td>
<td>1141.68</td>
<td>145</td>
<td>12.84</td>
</tr>
<tr>
<td>5</td>
<td>1.25% ZZ PF</td>
<td>360</td>
<td>801.19</td>
<td>1141.68</td>
<td>145</td>
<td>12.84</td>
</tr>
</tbody>
</table>

PF = Plastic fiber, ST = Straight, ZZ = Zigzag, and CA = Coarse Aggregate

2.3. Test Specimens and Test Procedures

For the compressive strength test, cylindrical specimens of 10 cm diameter and 20 cm height were used. Total 10 sets of specimens were cast, five sets for the destructive test and the other five sets for the non-destructive test. Three specimens were used for each set and the average value of compressive strength was used. Compressive strength for each specimen was determined by destructive and non-destructive Rebound Hammer test according to ASTM C39 and ASTM C805 respectively. Workability measurement was done according to ASTM C143 [15-17].

3. Result and Discussion

3.1. Result of Fresh Concrete Properties

There are many properties of fresh concrete. In this study, only the workability measurement is done. From Table 2, it can be seen that the addition of fibers decreases workability. The reduction of workability of concrete made with
zigzag fibers is a little bit more than concrete made with straight fibers. The finding confirms the results of a previous study of using shredded PET bottle waste as a partial substitution of natural aggregate [9]. During the test, segregation and bleeding of fresh concrete were also observed. Segregation can be defined as the separation of the constituents of a homogeneous mixture. Bleeding is a form of segregation in which some of the water in the concrete mix tends to rise to the surface of freshly placed concrete. The observation has shown that the fresh concrete mixture experiences no segregation and bleeding.

<table>
<thead>
<tr>
<th>% of CA Replacement</th>
<th>Slump value in cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain concrete</td>
</tr>
<tr>
<td>0</td>
<td>12.7</td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
</tr>
<tr>
<td>1.25</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2. Slump test

CA = Coarse Aggregate

3.2. Results of Hardened Concrete Properties

In this study, the compressive strength test (Destructive test) and Rebound Hammer test (Non-destructive test) were performed. From Tables 3 and 4, it can be said that the addition of fibers decreases the compressive strength of concrete because of poor bonding between plastic fibers and cement-sand paste. Poor bonding of plastic fiber occurs with the cement-paste due to the smooth surface of plastic fiber. 0.75 and 1.25% substitution of PET plastic straight fibers resulted in a decrease of 1.51 and 5.04% in the concrete’s destructive compressive strength from 19.84 MPa of standard concrete. The substitution of 0.75 and 1.25% PET plastic zigzag fibers resulted in a decline of 6.80 and 20.92% in the destructive compressive strength of concrete from 19.84 MPa of standard concrete. The substitution of 0.75 and 1.25% PET plastic zigzag fibers resulted in a decrease of 23.71 and 35.05% in the non-destructive compressive strength of concrete from 13.58 MPa of standard concrete. The substitution of 0.75 and 1.25% PET plastic straight fibers resulted in a decrease of 19.54 and 40.35% of the non-destructive compressive strength of concrete from 13.58 MPa of standard concrete. Rebound Hammer test is a non-destructive testing method of concrete that provides a convenient and rapid indication of the compressive strength of the concrete. The rebound hammer test method is based on the principle that the rebound of an elastic mass depends on the hardness of the concrete surface against which the mass strikes. Concrete with low strength and low stiffness will absorb more energy to yield in a lower rebound value. Based on the results obtained from the non-destructive compressive strength test, concrete with zigzag fibers absorbs more energy than concrete with straight fibers. It proves the previous study that usage of PET fibers provides ductility property [6]. Due to exposure of curing to the environment, weathering actions such as rain, sunlight variation, change in temperature, growth of organic content affect the hydration process of concrete and strength gaining process of concrete. Following previous studies, plastic fiber addition corresponds to a reduction of concrete's compressive strength [7-9].

<table>
<thead>
<tr>
<th>% of CA Replacement</th>
<th>Crushing Strength in MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain concrete</td>
</tr>
<tr>
<td>0</td>
<td>19.84</td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
</tr>
<tr>
<td>1.25</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3. Destructive compression test

CA = Coarse Aggregate

<table>
<thead>
<tr>
<th>% of CA Replacement</th>
<th>Crushing Strength in MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain concrete</td>
</tr>
<tr>
<td>0</td>
<td>13.58</td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
</tr>
<tr>
<td>1.25</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 4. Non-destructive compression test

CA = Coarse Aggregate
3.3. Failure Surface and Cracking of Concrete

The presence of fibers changes the mode of failure. The shear fracture occurs in plain concrete. The axial split occurs in concrete made with straight and zigzag fibers respectively. From Figure 3, it can be said that concrete with fibers shows some ductility property but plain concrete shows a brittle property. Splitting portion from concrete made with straight fibers is more than concrete made with zigzag fibers. Zigzag fibers in concrete hold some concrete portion during concrete failure and thus it provides the interlocking property with concrete. Usage of PET fibers avoids shear fracture by creating cracking. Concrete with zigzag fibers decreases more crack compared to concrete with straight fibers.

![Figure 3. (a) Plain concrete; (b) Concrete with zigzag fibers; (c) Concrete with straight fibers](image)

4. Conclusions

By observing results and discussions, the following conclusions are plotted:

- Overflow of water, the variation of sunlight, growth of organic content is found, as curing is exposed to the environment and these factors affect the hydration process and strength gaining process of concrete;
- The workability of concrete decreases by using plastic fibers;
- The presence of plastic fibers changes the mode of failure;
- The addition of fibers decreases the brittle property of concrete but increases the ductility of concrete;
- A low amount of plastic fibers should be used in concrete so that variation in compressive strength becomes negligible;
- Deformation of fibers in zigzag pattern provides the interlocking property with concrete;
- The use of plastic fibers decreases cracking. Zigzag fiber reduces more crack than straight fibers;
- Substitution of 0.75 and 1.25% of PET straight fibers give decreases in non-destructive compressive strength of 23.71 and 35.05% of the non-destructive compressive strength of the standard concrete (13.58 MPa), respectively. Substitution of 0.75 and 1.25% of PET zigzag fibers give decreases in non-destructive compressive strength of 39.54 and 40.35% of the non-destructive compressive strength of the standard concrete (13.58 MPa), respectively.

Substitution of 0.75 and 1.25% of PET straight fibers give decreases in destructive compressive strength of 1.51 and 5.04% of the destructive compressive strength of the standard concrete (19.84 MPa), respectively. Substitution of 0.75 and 1.25% of PET zigzag fibers give decreases in destructive compressive strength of 6.80 and 20.92% of the destructive compressive strength of the standard concrete (19.84 MPa), respectively.

5. Declarations

5.1. Data Availability Statement

Data sharing is not applicable to this article.
5.2. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.3. Acknowledgements

The authors would like to express their deepest gratitude to all concerned persons of the Department of Civil Engineering, Hajee Mohammad Danesh Science and Technology University for their valuable information and constructive suggestions. Acknowledgments are very due for Dr. Ismail Saifullah, Associate Professor, Department of Civil Engineering, Khulna University of Engineering & Technology, for his coordination and valuable suggestions about this research.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Performance of Retrofitted Square Reinforced Concrete Column using Wire Mesh and SCC Subjected to Cyclic Load

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Received 03 January 2021; Revised 24 March 2021; Accepted 31 March 2021; Published 01 April 2021

Abstract

One way to restore or increase the strength of the structure against earthquakes is to use retrofit method and wire mesh is a material that has high prospects as retrofit material. The purpose of this study was to examine the use of wire mesh as a retrofit material on reinforced concrete columns burdened with cyclic loads. In this study, testing of 3 square column samples of reinforced concrete with dimensions of 300 × 300 mm. The first specimen is fully retrofit on the entire cross-section of the column, the second specimen is retrofitted on the plastic hinge area of the column and the third specimen is a control column without retrofit. In the first and second specimens were retrofitted with wire mesh size M6 using SCC which was then tested with a cyclic load using displacement control method based on the provisions stipulated in the Indonesian Standard SNI 7834:2012. From the test results and analysis results, it was found that the capacity and ductility of displacement in retrofit specimens increased significantly compared to specimens that were not retrofit. In addition, the decrease in stiffness in retrofit specimens was smaller than in non-retrofit specimens. As for the value of energy dissipation in fully retrofit specimens and in retrofit on the plastic hinge area is almost close. Based on these conditions, the use of wire mesh size M6 and SCC can be used as retrofit material on the column that is burdened with cyclic load.

Keywords: Retrofit; Wire Mesh; SCC; Strength, Ductility; Stiffness; Dissipation Energy.

1. Introduction

An earthquake is a potential energy release event from the earth's stomach that then radiates to the earth's surface in the form of earthquake waves caused by activity from the movement of tectonic plates that until today have not been predicted when it will happen, where it is located and how much energy will be released. Earthquakes cause thousands of people to die every year, either directly or indirectly. In addition, earthquakes not only cause damage to structures, but cause gas explosions, spark fires and in recent years, earthquakes have resulted in the loss of many lives in Japan, China and Indonesia [1]. The main problem that occurs due to earthquakes is the occurrence of damage and failure on building structures, especially those designed using building regulations in the 1970s because it has a low ductility in the column, making it very vulnerable to earthquakes and in recent decades various methods of...
strengthening and repairing columns have been developed to obtain columns that meet the requirements of strength, ductility and durability [2]. To increase the ductility of reinforced concrete columns can be achieved through the installation of repeating details properly and correctly, the use of confinement or the installation of jackets on the plastic hinge area of the column [3].

Retrofit installation on the column aims to increase the strength, deformability, ductility and stiffness of the column, where since this method was developed various types of materials have been studied using various parameters and based on the material used this method can be classified based on the use of concrete jackets, steel jackets, ferrocement laminated jackets, and FRP confinement [4] and currently fiber reinforced polymer (FRP) composite is the most advanced material and promising as a material for retrofit on the structure because it has a high tensile capability, but also reveals exceptional properties such as high durability, stiffness, damping property, flexural strength, and resistance to corrosion, wear, impact, and fire [5]. In addition to the use of one type of material for retrofit, some researchers also use a combination of several materials such as using basalt and FRP as a confinement material on square columns of reinforced concrete with inadequate transverse reinforcement [6], the use of a combination of self-compacting concrete, CFRP filled and steel tubes on reinforced concrete square columns tested with constant cyclic and axial loads [7] and the use of NSM GFRP reinforcing and steel bars on columns modeled before the 1970s using plain bars and different lap splices [8].

In general, wire mesh is a material with specifications that vary widely around the world, this is because wire mesh is made from a variety of metal-based materials, with different shapes and dimensions depending on the importance of their use. From the results of Kumar and Vatel's research that the use of stainless steel wire mesh on circular columns tested with axial loads resulted in a significant increase in strength [9]. According to Kadir, et.al, that the use of wire mesh as a retrofit material is based on the consideration that wire mesh is also flexible enough in forming a confinement pattern and the installation of wire mesh with a certain number of layers can produce a significant amount of ductility value on a column that is burdened with cyclic load, and can increase the strength of the column shear [10].

In Indonesia wire mesh is a widely known construction material with the term woven iron with diameters ranging from 4 mm to 12 mm. Research on the use of wire mesh in this case woven iron is still very minimal and has not been done much. Therefore, in this study, experimental studies were conducted using wire mesh or woven iron diameter 6 mm (M6) combined with SCC as retrofit material on columns burdened with cyclic load. The parameters of the study consisted of strength, ductility, stiffness and energy dissipation capacity, with the hope that the use of wire mesh or woven iron can be used as an alternative retrofit material in lieu of retrofit materials from fiber to be applied to earthquake-prone building construction, especially remote areas in Indonesia. Furthermore, this paper describes experimental methods that include the dimensions of test specimens, setting up and testing methods, experimental results and analysis results that show that the use of a combination of wire mesh and SCC can be used as retrofit materials on columns. The flow of research conducted in this study is presented in Figure 1.

![Figure 1. Research methodology flowchart](image)

2. Experimental Methods

2.1 Specimen Test Dimensions

In this study, the test specimen used was a square column of reinforced concrete as many as 3 specimens consisting of a retrofitted column filled with wire mesh and SCC along the column body (KR01), a column that was retrofit with wire mesh and SCC in the plastic hinge area (KR02) and columns that were not retrofitted with wire mesh and SCC.
(KK). The three specimens of reinforced concrete columns are made of normal concrete with a quality \( f'_c \) of 25 MPa with the same dimensions. After the specimen is 14 days old, in specimens KR01 and KR02 are retrofit using wire mesh in the form of woven iron diameter 6 mm (M6) which has a grid dimension of 150×150 mm. The retrofit coating is 50 mm thick and uses a quality SCC \( f'_c \) of 25 MPa. For details of specimen dimensions and reinforcement used are presented in Figure 2 and Table 1.

![Figure 2. Detail specimens dimensions](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column Dimensions (mm)</th>
<th>Column Height (mm)</th>
<th>Column Reinforcement (mm)</th>
<th>( f'_c ) (MPa)</th>
<th>( A_d/A_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>KK</td>
<td>300 × 300</td>
<td>1465</td>
<td>8D13 ( \Theta 8 - 150 )</td>
<td>25</td>
<td>0.0118</td>
</tr>
<tr>
<td>KR01</td>
<td>300 × 300</td>
<td>1465</td>
<td>8D13 ( \Theta 8 - 150 )</td>
<td>25</td>
<td>0.0118</td>
</tr>
<tr>
<td>KR02</td>
<td>300 × 300</td>
<td>1465</td>
<td>8D13 ( \Theta 8 - 150 )</td>
<td>25</td>
<td>0.0118</td>
</tr>
</tbody>
</table>

### 2.2. Setting Up and Testing Methods

Specimen testing is carried out after the specimen is 28 days old by placing the specimen in a predetermined position as shown in Figure 3.
The specimen was then tested with cyclic load based on displacement control method, where the amount of deformation given and the number of cycles is adjusted to the loading pattern referring to SNI 7834:2012 [11], as shown in Figure 4. During testing, three full cycles must be applied to each drift ratio, consisting of phase 1 which is the primary cycle and phases 2 and 3 which are stabilization cycles. During the test, data recorded on logger and computer data were taken, as well as visual observation of the cracks that occurred in the specimen.

**Figure 4. Loading pattern on specimen**

### 3. Experimental Result and Discussion

#### 3.1. Load, Drift Curve and Cracks

In KK specimens, the resulting hysteresis loops curve shape is not very fat and the decrease in stiffness is relatively insignificant as presented in Figure 5. At compressive loading, the KK test specimen curve experienced a significant increase in load up to a drift ratio of 2.2% and after that the curve experienced an insignificant increase result and tended to flatten to a drift ratio of 3.5%. While in the tensile loading, the KK specimen curve experienced a significant increase in load up to a drift ratio of 2.2% and after that the curve experienced an insignificant increase and tended to flatten to a drift ratio of 3.5% which is the minimum drift, without experiencing a decrease in strength in the specimen.

The relationship between load and deflection in retrofit column specimen KR01 due to working cyclic load is described in the form of hysteresis loops curve as presented in Figure 6. At compressive loading, the KR01 specimens curve experienced a significant increase in load to a drift ratio of 2.75% and after that the curve experienced an insignificant increase and tended to flatten to a drift ratio of 5.73%. In tensile loading, the KR01 specimen test curve experienced a significant increase in load up to a drift ratio of 3.5% and after that the curve experienced an insignificant increase and tended to flatten to a drift ratio of 5.73%.

The relationship between load and deflection in KR02 specimens due to working cyclic loads can be described in the form of hysteresis loops curves as presented in Figure 7. At compressive loading, the KR02 specimen curve experienced a significant increase in load to a drift ratio of 2.75% and after that the curve experienced an insignificant increase and tended to flatten to a drift ratio of 5.73%. In tensile loading, the KR02 test object curve experienced a significant increase in load up to a drift ratio of 3.5% and after that the curve experienced an insignificant increase and tended to flatten to a drift ratio of 5.73%.

**Figure 5. Hysteresis curve specimen KK**
As a result of the increase in cyclic load in the three specimens caused a gradual change in behavior from linear to non linear gradient behavior, thus exhibiting an inelastic behavior upon reaching the post yielding zone that automatically led to lateral stiffness changes in the specimen. Until the end of the test on compressive and tensile direction loading, the strength of KR01 and KR02 specimens has a higher strength compared to KK specimens and both specimens have not collapsed. In addition, all specimens meet the required minimum drift criteria of 3.5%.

In the first crack KK specimen occurred at a drift ratio of 0.75% due to a tensile load of 8,550 kN and subsequent cracks occurred due to a compressive load of 9,890 kN. The initial crack position is in the area of potential plastic hinge with a distance of 30 cm from the column legs. As a result of the increase in the number of cycles, the number of cracks continues to grow in the area of the column plastic hinge which then extends to the middle to the top of the specimen to a drift ratio of 1.75%. While until the end of testing at a drift ratio of 3.5% does not appear new cracks and the only addition of crack width.

In specimen KR01 the first crack occurred at a drift ratio of 0.75% due to a tensile load of 11,100 kN and the initial crack position was in the area of plastic hinge with a distance of 33 cm from the column legs. The subsequent crack occurred due to a compressive load of 17,850 kN and continued to increase in the area of the column plastic hinge which then spread to the center to the top of the specimen up to a deviation ratio of 1.75%. Until the end of the test at a drift ratio of 5.73% did not appear new cracks and the addition of the width of cracks that occurred was not as significant as in specimens KK.

In the KR02 specimen the first crack occurred at a drift ratio of 0.75% due to a tensile load of 10,560 kN and subsequent cracks occurred due to a compressive load of 15,525 kN. The initial crack position is in the area of the plastic hinge with a distance of 31 cm from the column legs. As a result of the increase in the number of cycles, the number of cracks continues to grow in the area of the column plastic hinge which then extends to the middle to the top of the specimen to a drift ratio of 1.75%. Until the end of the test at a drift ratio of 5.73% did not appear new cracks and the addition of crack width was not as significant as in KK specimens.
In the three specimens the crack pattern that occurs is almost the same, resulting in the same shear cracking pattern with the direction in which the cyclic load works. As for the addition of new cracks only to the drift ratio of 1.75%, where this is due to the mechanism of plastic hinge in the legs of the column has formed and longitudinal reinforcement attached to the area of plastic hinge has begun to yield.

3.2. Degradation of Strength in Specimens

For columns burdened with cyclic loads, SNI 7834:2012 [11] requires that the column should not experience strength degradation (D), where this condition can occur if the peak force (Pf) working is less than 75% of the maximum lateral load (Emax) in the same loading direction and the analysis results are presented in Table 2.

Table 2. Requirements for degradation of strength in specimens

<table>
<thead>
<tr>
<th>Specimen Test</th>
<th>Lateral Force (kN)</th>
<th>Emax (kN)</th>
<th>0.75Emax (kN)</th>
<th>Pf (kN)</th>
<th>D = 1 - Pf/Emax (%)</th>
<th>Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>KK Compressive</td>
<td>24.98</td>
<td>18.735</td>
<td>24.80</td>
<td>0.721</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>KK Tensile</td>
<td>27.20</td>
<td>-20.400</td>
<td>26.88</td>
<td>1.176</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>KR01 Compressive</td>
<td>36.64</td>
<td>27.480</td>
<td>34.90</td>
<td>4.749</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>KR01 Tensile</td>
<td>37.50</td>
<td>-28.125</td>
<td>37.00</td>
<td>1.333</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>KR02 Compressive</td>
<td>35.40</td>
<td>26.550</td>
<td>33.00</td>
<td>6.780</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>KR02 Tensile</td>
<td>36.00</td>
<td>-27.000</td>
<td>35.00</td>
<td>2.778</td>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

From the results of the analysis in Table 2 shows that all specimens meet the requirements of strength degradation, since the peak force value (Pf) in all specimens is greater than 75% of the maximum lateral load value that occurs.

3.3. Ductility

Ductility is the ability of the structure to formalize in elastically without experiencing a significant reduction in strength before it reaches collapse and the properties of structural ductility describe the amount of energy capable of being absorbed by the structure [12], where the value of displacement ductility occurs, can be expressed as a comparison between deflection at the time of ultimate condition (Δu) and deflection at the time of the first yield (Δu) occurred [13] and the results of ductility analysis on all specimens are shown in the Figure 8.

![Figure 8. Displacement ductility value in specimens](image)

Based on the results of the analysis, the value of displacement ductility in KK specimens due to compressive load of 2.074 and due to tensile load of 2.077. In the KR01 test specimen, due to the compressive load was 3.165 and the tensile load was 3.144. While in KR02 specimen, the value of displacement due to compressive load is 3.275 and due to tensile load of 3.275 and according to SNI 1726:2019 [14], that for the value of ductility more than 1.5 is included in the performance level of the partial ductility structure. If the value of displacement ductility in KR01 and KR02 specimens compared to the ductility value in KK specimens, then the value of displacement ductility in KR01 specimens increased by 52.6% due to compressive load and 51.4% due to tensile load. While the value of displacement ductility in KR02 specimens increased by 57.9% due to compressive load and 57.7% due to tensile load.
3.4. Stiffness

For structures experiencing cyclic loads, stiffness is defined as the slope of the line connecting the peaks of the maximum load in the positive and negative direction of the load curve and deflection [15], where the results of the analysis of the comparison of stiffness values in all specimens due to the compressive load are presented in Figure 9.a and tensile load are presented in Figure 9.b.

Due to the compressive load there was a decrease in stiffness in KK specimens by 32.15%, in KR01 specimens by 22.54% and in KR02 specimens by 21.74% at the end of loading. Meanwhile, due to the tensile load there was a decrease in stiffness in KK specimens by 52.15%, in KR01 specimens by 35.95% and in KR02 specimens by 39.89% at the end of loading.

![Figure 9. Stiffness of the specimen due to the (a) compressive load; and (b) tensile load](image)

In addition, in specimens KR01 and KR02 the decrease in stiffness relatively did not occur, so it is likely that no pinching effect occurred and more led to more stable conditions with higher energy dissipation ability.

3.5. Initial Stiffness and Degradation of Stiffness

According to SNI 7834:2012 [11], that test specimen must meet the initial stiffness requirement, where the test specimen must reach the minimum lateral resistance value ($E_n$) before its drift ratio of 2% exceeds the value consistent with the permissible drift ratio limit. The results of the initial stiffness analysis are presented in Table 3. In addition, SNI 7834:2012 [11] requires that for structures or specimens designed against earthquakes must have sufficient stiffness degradation at the time of an earthquake or cyclic load. Test specimens are considered to have sufficient stiffness degradation, if the stiffness of the line connecting the intersection point -0.0035 to the drift ratio of +0.0035 is not less than 0.005 of the initial stiffness value ($K_0$) and the analysis results are presented in Table 4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cyclic Force</th>
<th>$E_n$ (kN)</th>
<th>$\Delta_0$ (mm)</th>
<th>$h$ (mm)</th>
<th>$r_1 = \Delta_0/h$ (%)</th>
<th>$r_2$ (%)</th>
<th>$r_1 &lt; (1 + 0.02).r_2$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KK</td>
<td>Compressive</td>
<td>20,000</td>
<td>12,400</td>
<td>1460</td>
<td>0.849</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>23,000</td>
<td>13,000</td>
<td>1460</td>
<td>0.890</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
<tr>
<td>KR01</td>
<td>Compressive</td>
<td>30,000</td>
<td>12,750</td>
<td>1460</td>
<td>0.873</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>26,400</td>
<td>12,760</td>
<td>1460</td>
<td>0.874</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
<tr>
<td>KR02</td>
<td>Compressive</td>
<td>27,600</td>
<td>12,780</td>
<td>1460</td>
<td>0.875</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>24,500</td>
<td>12,780</td>
<td>1460</td>
<td>0.875</td>
<td>3.5</td>
<td>Accepted</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cyclic Force</th>
<th>$K_0$ (kN/mm)</th>
<th>$K'$ (kN/mm)</th>
<th>$r_1 = K'/K_0$</th>
<th>Requirements $r_1 &gt; 0.05$</th>
</tr>
</thead>
<tbody>
<tr>
<td>KK</td>
<td>Compressive</td>
<td>3.021</td>
<td>2.964</td>
<td>0.981</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Tensile</td>
<td>2.105</td>
<td>2.343</td>
<td>1.113</td>
<td>Accepted</td>
</tr>
<tr>
<td>KR01</td>
<td>Compressive</td>
<td>3.699</td>
<td>2.219</td>
<td>0.600</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Tensile</td>
<td>2.459</td>
<td>2.219</td>
<td>0.902</td>
<td>Accepted</td>
</tr>
<tr>
<td>KR02</td>
<td>Compressive</td>
<td>3.627</td>
<td>2.109</td>
<td>0.582</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>Tensile</td>
<td>2.096</td>
<td>2.109</td>
<td>1.006</td>
<td>Accepted</td>
</tr>
</tbody>
</table>
In Table 3 the minimum lateral resistance value \( (E_n) \) is taken from the lateral force at the time of yielding \( (P_y) \) and the minimum lateral drift is taken from the drift value at the time of yielding \( (\Delta_\text{y}) \). While the value of lateral resistance drift ratio \( (r_1) \) is a comparison between the value of drift at the time of yielding \( (\Delta_\text{y}) \) and the height of the specimen \( (h) \) and \( r_2 \) is the minimum drift of 0.035 \%(\%). From the results of the analysis, it can be concluded that all specimens meet the initial stiffness requirements, because the value of the lateral resistance drift ratio \( (r_1) \) is less than the requirements specified in SNI 7834:2012 [11]. From the results of the analysis in Table 4 showed that all specimens meet the requirements of stiffness degradation, where the \( r_1 \) ratio value in all specimens is greater than 0.05.

3.6. Energy Dissipation

The main purpose of retrofit is to increase the capacity of structural elements, where capacity can increase in the event of an increase in energy dissipation capacity at the time of an earthquake, without a significant decrease in strength. Energy dissipation capacity is an important parameter for planned structures with earthquake loads that have long earthquake re-periods and energy dissipation values in a single cycle \( (E_D) \) can be calculated based on the area \( (A) \) of the relationship between lateral force occurring, with deformation in the form of a closing curve called hysteresis loops [16]. For energy dissipation and accumulative energy dissipation values on all specimens are used values from the first cycle which is the primary cycle, as presented in Figure 10.a and Figure 10.b.

![Figure 10. (a) Dissipation energy; and (b) Accumulative dissipation energy](image)

From the energy dissipation comparison graph in Figure 10.a, it appears that the energy dissipation value in KR01 and KR02 specimens is greater than the energy dissipation value in KK specimens, while the energy dissipation value in KR02 looks higher than KR01 at drift ratio of 3.5 to 4.48% and decreases again at a drift ratio of 5.73%. While accumulatively KR02 specimens from Figure 10.b, are able to provide more energy than KK and KR01 specimens. When compared with the accumulative energy dissipation value in KK specimens at a drift ratio of 3.5\%, there was an increase in energy dissipation by 23.32\% in KR01 specimens and 26.30\% in KR02 specimens. In all specimens the
accumulative value of energy dissipation tends to increase towards increased drift levels, but the amount of energy dissipation tends to decrease for each repeat cycle at each drift level. This condition can be caused by the development of cracks at the same level of drift relatively constant, so that no new cracks are formed but only the addition of the width of the cracks at the same crack location.

To control the stability of the structure system at the maximum drift level due to the cyclic load working on the specimen, SNI 7834:2012 [11] requires that the relative energy dissipation ratio (β) should not be less than 0.125 calculated based on the third cycle at the end of loading. The results of analysis of relative energy dissipation ratio in specimens are presented in Figure 11.

![Figure 11. Minimum criteria relative energy dissipation ratio](image)

The relative energy dissipation ratio value in Figure 11, calculated from the ratio of actual energy dissipation value to ideal energy dissipation value and based on the analysis results, it can be concluded that the ratio of relative energy dissipation (β) in the three specimens meets the minimum requirements in SNI 7834:2012 [11], so that the three test specimens still have the ability to maintain their stability before collapse.

### 4. Conclusion

From the test results can be concluded the capacity of the column in receiving cyclic loads that work on the model of the retrofitted column is filled with wire mesh and SCC along the column body and on the model of the column that is retrofitted with wire mesh and SCC on the plastic hinge area, resulting in a force that is close to. Both models have a greater capacity in receiving cyclic load compared to column models that are not retrofitted with wire mesh and SCC. The ductility value in the retrofitted column model filled with wire mesh and SCC along the column body increased by 52.6% on compressive loading and 51.4% on tensile loading. In the column model that was retrofit with wire mesh and SCC in the plastic hinge area, the ductility value increased by 57.9% on compressive loading and 57.7% on tensile loading, when compared to the ductility value in the column model that was not retrofitted with wire mesh and SCC.

As for the decrease in the value of stiffness occurs in specimens that are not retrofit with wire mesh and SCC. The capacity of energy dissipation and accumulative energy dissipation in the retrofitted column model is full of wire mesh and SCC along the column body and the column model is retrofitted with wire mesh and SCC on the plastic hinge area, almost the same and larger when compared to the column model that is not retrofitted with wire mesh and SCC.

From the results of this study can conclude that the use of 1 layer of wire mesh in this case woven iron with a diameter of 6 mm (M6) combined with quality SCC (f'_c) 25 MPa can increase the strength capacity of the column, increase the value of ductility and stiffness, and have the ability to dissipate energy better so that this combination of materials can be used as retrofit materials on reinforced concrete square columns burdened with cyclic loads. Considering this research is an early study, so it is expected that in the future there will be many further research. For future research opportunities, it is necessary to conduct research with the parameters of the use of geometry variations, column dimensions, longitudinal reinforcement diameter, transverse reinforcement diameter, wire mesh size, thick variation of retrofit layer, number of wire mesh layers, and addition of anchors mounted on plastic hinge area. In addition, it is necessary to conduct research with a combination of cyclic load and constant a axial load, considering that in practice in the field the column structure also carries an a axial load, as well as a combination of the use of wire mesh as a reinforcement in the replacement of confinement and the use of wire mesh as retrofit.
5. Declarations

5.1. Data Availability Statement
The data presented in this study are available in article.

5.2. Funding
This paper and the research behind it would not have been possible without the exceptional support of LPDP Ministry of Finance Republic Indonesia through the BUDI DN Scholarships, and Earthquake Research Laboratory University of Hasanuddin Makassar.

5.3. Conflicts of Interest
The authors declare no conflict of interest.

6. References


Numerical Study of Laterally Loaded Piles in Soft Clay Overlying Dense Sand

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Received 09 January 2021; Revised 16 March 2021; Accepted 23 March 2021; Published 01 April 2021

Abstract

This paper presents the results of three dimensional finite element analysis of laterally loaded pile groups of configuration 1×1, 2×1 and 3×1, embedded in two-layered soil consisting of soft clay at liquid limit overlying dense sand using Plaxis 3D. Effects of variation in pile length (L) and clay layer thickness (h) on lateral capacity and bending moment profile of pile foundations were evaluated by employing different values of pile length to diameter ratio (L/D) and ratio of clay layer thickness to pile length (h/L) in the analysis. Obtained results indicated that the lateral capacity reduces non-linearly with increase in clay layer thickness. Larger decrease was observed in group piles. A non-dimensional parameter Fx ratio was defined to compare lateral capacity in layered soil to that in dense sand, for which a generalized expression was derived in terms of h/L ratio and number of piles in a group. Group effect on lateral resistance and maximum bending moment was observed to become insignificant for clay layer thickness exceeding 40% of pile length. For a fixed value of clay layer thickness, lateral capacity and bending moment in a single pile increased significantly with increase in pile length only up to an optimum embedment depth in sand layer which was found to be equal to three times pile diameter and 0.21 times pile length for pile with L/D 15. Scale effect on lateral capacity has also been studied and discussed.

Keywords: Laterally Loaded Piles; Lateral Displacement; Lateral Capacity; Layered Soil; Bending Moment; Soft Clay; Dense Sand; Finite Element Analysis.

1. Introduction

Study of behavior of laterally loaded pile foundations embedded in different subsoil strata has always been a challenging field for the researchers since the commencement of the use of these foundations in various geotechnical projects. After the pioneer research on laterally loaded piles done by Matlock and Reese (1962) [1] and Broms (1964, 1965) [2–4], a vast research has been reported based on theoretical as well as practical studies conducted on laterally loaded piles embedded in homogeneous sands and clays using various experimental and numerical approaches. Majority of the reported initially studies used p-y method based on subgrade reaction approach, but with further advancement in the research, different numerical methods based on elastic continuum approach have been reported. Ashour and Norris (2000) recommended the use of strain wedge model rather than traditional p-y method to

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http://dx.doi.org/10.28991/c ej-2021-03091686

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characterize influences of pile stiffness, pile cross sectional shape, pile head fixity and pile head embedment and soil properties on p-y curves [5]. Patra and Pise (2001) conducted an experimental study on model pile groups in dry sand and proposed analytical methods to predict ultimate lateral capacity of single and group piles based on pile friction angle, length to diameter ratio, soil properties, pile configuration and spacing of piles in group [6]. Ilyas et al. (2004), through a series of centrifuge model tests on laterally loaded pile groups in normally consolidated and overconsolidated clays, reported a decrease in group efficiency with increase in number of piles in the group and shadowing effect phenomenon showing that the front piles experience larger load and bending moment than the trailing piles [7]. Krishnamurthy et al. (2005) studied the behavior of laterally loaded single and group piles using non-linear finite element method incorporating a hypoelasticity constitutive model to model soil behavior and reported the effect of various parameters including pile spacing, direction of load, arrangements of piles in a group and thickness of pile cap on lateral load capacity of each pile in a group [8]. Phanikanth and Choudhary (2014) performed parametric analysis of single pile with floating tip in cohesionless soil using elastic continuum method and modulus of subgrade reaction approach and then proposed algebraic equations for free headed and fixed headed floating tip piles [9].

Reasonable application of finite element method or finite difference method based analytical software tools like Plaxis 3D and FLAC 3D to analyze the behavior of pile foundations has also been reported in various studies. Sivapriya and Gandhi (2013) conducted experimental model tests and numerical study with Plaxis 3D to evaluate behavior of a laterally loaded single pile in sloping clay. Based on the analysis results, non-dimensional charts were prepared for lateral capacity of pile in sloping ground [10]. Gouw (2017) reported the results of 3D finite element analysis of pile groups of configuration 5×5 and 9×9 using Plaxis 3D. Effects of pile spacing, number of piles in a group and magnitude of pile head lateral movement on pile group lateral efficiency were reported in the study [11]. Abhishek and Sharma (2019) reported the results of numerical analysis of uplift performance of granular pile anchors in expansive soils using Plaxis 3D by applying prescribed displacement of 10% of pile diameter at the center of pile top. Effect of pile length, diameter, number of pile group anchors at different spacing and modulus of elasticity on uplift capacity was studied [12]. Choi et al. (2018) investigated lateral behavior of pile foundations socketed into bedrocks using FLAC 3D and studied the influence of bedrock depth on pile deflection profiles, bending moment distribution in piles and p-y curves. It was reported that effect of bedrock gradually decreases with increase in bedrock depth and disappears at bedrock depth of 10 times diameter or more [13].

Many studies based on different analytical approaches have been reported on laterally loaded piles embedded in layered strata. Yang and Liang (2006) proposed a numerical solution using beam on an elastic foundation model for analysis of laterally loaded piles incorporating a variation in soil stiffness in layered soil [14]. Li and Gong (2008) developed an analytical method based on basic structural mechanics to obtain equations for deflection, bending moment and soil reaction in fixed and free head laterally loaded single pile in layered soils [15]. Hirai (2012) used Winkler model approach for the analysis of single pile and pile groups subjected to vertical and lateral load in non-homogeneous soils [16]. Ai et al. (2013) proposed a theory based on boundary element method for static analysis of laterally loaded piles in multilayered transversely isotropic soils [17]. Gupta and Basu (2017) developed a continuum-based method to analyze laterally loaded piles in multilayered heterogeneous elastic soil with soil modulus varying linearly or non-linearly with depth in soil layers [18]. Gerolymose et al. (2020) derived analytical expressions for failure envelope of piles subjected to horizontal and moment loading and verified the derived relations through numerical methods based on Winkler analysis and continuum mechanics analysis and also validated the results through experimental model tests. The effects of parameters like mesh density, soil strength properties, interface non-linearities and soil constitutive models on post failure response of pile –soil system was investigated [19]. Gupta and Basu (2020) developed a method of analysis for short rigid laterally loaded piles in multi-layered elastic soil using the variational principals of mechanics and obtained equilibrium equations for pile and soil displacements using principal of virtual work which were solved using an iterative algorithm. The soil resistance against pile movement was related to the soil elastic constants and pile head displacement and rotation were calculated through the developed analytical method [20].

Stiffness of clayey soil varies largely with its plasticity, consistency and water content. Highly plastic soft clay at liquid limit possessing only a marginal stiffness is usually encountered in the field constructions near coastal areas. However, research reported on laterally loaded piles embedded in clayey soil or multilayered soil deposits mostly consider stiff clays or soils possessing a considerable amount of stiffness modulus [10, 13, 17]. A limited work has been reported on laterally loaded piles embedded in soft clay [21, 22]. However, research data on behavior of laterally loaded piles in layered soil consisting of soft clay at liquid limit overlying dense sand is still meager. Considering this issue, an attempt has been made through this paper to present the results of a finite element study conducted on laterally loaded single and group piles embedded in two layered soil consisting of a layer of soft clay with water content equal to liquid limit overlying a layer of dense sand. Finite element analysis was performed using Plaxis 3D version 2012 to study lateral load capacity of piles in different arrangements of layer thickness and pile dimensions. The present analysis was extended from the experimental model testing by Kaur et al. (2021) [23]. Firstly, finite element analysis was performed on small models with dimensions and soil properties similar to laboratory model tests.
to validate the numerical models. Numerical analysis was then performed on large prototype models with different scales. In this paper, results obtained from finite element analysis of prototype models are presented and effects of variation in pile length and clay layer thickness on lateral capacity and bending moment profile of pile foundations are discussed. Scale effect on lateral capacity evaluated from the obtained results is also presented in this paper.

Section 2 briefly discusses the research methodology which includes development of finite element models, selection of various parameters including soil properties, thickness of soil layers, pile dimensions, pile group arrangements and different scales of the numerical models. Section 3 presents the validation record of finite element modeling by comparing the numerical analysis results to the experimental model test results. Section 4 presents results and discussions which include effect of variation of clay layer thickness and pile length on lateral capacity and bending moment profile of piles and effect of scale of numerical model on pile lateral capacity. Conclusions are drawn in section 5. The research process followed in this study is shown in Figure 1.

2. Finite Element Modeling

2.1. Generation of Prototype Soil Model

A prototype model with size equal to 20 times size of experimental model, consisting of a two-layered soil strata of dimensions 40×40×20 m was generated for the finite element analysis in which upper layer consisted of CH clay at liquid limit and lower layer was of dense sand with relative density of 80%. Three different types of pile arrangements employed in the analysis were single pile (1×1), two piles in a row (2×1) and three piles in a row (3×1) with pile spacing equal to three times pile diameter. Figure 2 shows the general layout of the model generated with pile group 3×1.

In Plaxis 3D, 10-node tetrahedral elements are used to discretize the soil elements in the finite element mesh. In addition to the soil elements, 3-node line elements are used for beams, 6-node elements for plates and 12-node interface elements of zero thickness are used to model soil-pile interaction. Mohr-Coulomb model was used to simulate the soil behavior. This model requires five parameters to define strength and stiffness characteristics of soil which include stiffness (E'), Poisson’s ratio (ν'), cohesion (c'), friction angle (φ') and dilatancy angle (ψ') [24]. Properties assigned to the clay layer and the sand layer are listed in Table 1. Assigned values of unit weight of both the soils and stiffness and shear parameters of dense sand were determined through values of unit weight of soils in both the layers, stiffness and shear parameters of dense sand were determined through laboratory model tests. Dilatancy angle for both the soils was selected as per recommendations by Bolton i.e. for sands with φ ≥ 30°, ψ ≈ φ-30° and for clays, ψ ≈ 0 [25]. To avoid complications in the analysis, Plaxis recommends to use a small value of cohesion for sand greater than 0.2 kN/m², so a value of 0.3 kN/m² for cohesion of sand layer was used in the analysis. Stiffness and shear
parameters of soft clay at liquid limit were selected as per available literature [26-29]. Obrzud and Truty (2018) [26] compiled typical values of stiffness modulus for clayey soils from reports of Kezdi (1974) [27] and Prat et al. (1995) [28] that stiffness modulus for CH soil of very soft to soft consistency varies from 0.35 to 4 MPa. Sridharan (1991) reported that soils at liquid limit possess a definite but small shearing strength of order of 15-30 g/cm² [29]. The stiffness modulus of 0.35 MPa i.e. 350 kN/m² and shear strength of 15 g/cm² i.e. 1.472 kN/m² were adopted after performing various trial numerical analyses with different values of these parameters in small scale numerical modeling.

![Figure 2. General layout of the finite element model](image)

### 2.2. Modeling of Piles and Pile Cap

Model piles were created using embedded pile option. An embedded pile is composed of beam elements and embedded interface elements to interact with soil elements at its surface and foot. Dimensions of the pile used in numerical prototype modeling were scaled from dimensions of model steel pipe piles used in experimental tests using a scale of 20. Properties assigned to the model pile are given in Table 2. Pile cap was modeled as a concrete cap of M40 grade using the plates option which is used to model thin structures with a significant flexural rigidity. Properties assigned to the pile cap are given Table 3.

#### Table 1. Soil Properties used in numerical analysis

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
<th>Top Layer Clay</th>
<th>Bottom Layer Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Model</td>
<td></td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td></td>
</tr>
<tr>
<td>Drainage Type</td>
<td></td>
<td>Drained</td>
<td>Drained</td>
<td></td>
</tr>
<tr>
<td>Dry unit weight, γ\text{sat}</td>
<td>kN/m³</td>
<td>15.3</td>
<td>17.7</td>
<td></td>
</tr>
<tr>
<td>Saturated unit weight, γ\text{sat}</td>
<td>kN/m³</td>
<td>18.74</td>
<td>18.4</td>
<td></td>
</tr>
<tr>
<td>Stiffness, E</td>
<td>kN/m²</td>
<td>350</td>
<td>39000</td>
<td></td>
</tr>
<tr>
<td>Poisson’s Ratio, ν'</td>
<td></td>
<td>0.4</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Cohesion, c'</td>
<td>kN/m²</td>
<td>1.472</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Angle of Friction, φ'</td>
<td>°</td>
<td>0</td>
<td>34.6</td>
<td></td>
</tr>
<tr>
<td>Dilatancy Angle, ψ</td>
<td>°</td>
<td>0</td>
<td>4.6</td>
<td></td>
</tr>
<tr>
<td>Strength Reduction Factor, R\text{inter}</td>
<td></td>
<td>0.67</td>
<td>0.67</td>
<td></td>
</tr>
</tbody>
</table>

#### Table 2. Properties of model pile

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity, E</td>
<td>kN/m²</td>
<td>150x10⁶</td>
</tr>
<tr>
<td>Unit Weight, γ</td>
<td>kN/m³</td>
<td>73.37</td>
</tr>
<tr>
<td>Pile Type</td>
<td></td>
<td>Circular Tube</td>
</tr>
<tr>
<td>Outer Diameter, D</td>
<td>m</td>
<td>0.254</td>
</tr>
<tr>
<td>Wall Thickness, t</td>
<td>m</td>
<td>0.008</td>
</tr>
<tr>
<td>Embedded Length, L</td>
<td>m</td>
<td>7.62</td>
</tr>
</tbody>
</table>
Table 3. Properties of pile cap

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, (d)</td>
<td>m</td>
<td>0.5</td>
</tr>
<tr>
<td>Unit Weight, (\gamma)</td>
<td>kN/m³</td>
<td>25</td>
</tr>
<tr>
<td>Modulus of Elasticity, (E = 5000\sqrt{f_{ck}})</td>
<td>kN/m²</td>
<td>31.6x10⁶</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td></td>
<td>0.15</td>
</tr>
</tbody>
</table>

2.3. Analysis Scheme

Finite element mesh was generated after completing the model. After that calculations were performed in phases through stage construction mode. In first phase, initial stresses were calculated using \(K_0\) procedure. Soil in both the layers and various structural elements were activated in next phases. Model piles were subjected to point displacement equal to 20% of pile diameter acting at pile head. Prescribed point displacement was activated in the last phase of calculation process. The lateral load (\(F_x\)) corresponding to the applied displacement was obtained using charts of load – displacement curves at the selected nodes.

2.4. Parametric Study

A number of parameters were varied to analyze their effect on lateral response of pile groups. While keeping the diameter of the piles constant, pile length was varied to use various values of length to diameter ratio (L/D) in the analysis. Clay layer thickness was varied from 0 to \(L\) i.e. equal to pile length and hence the value of \(h/L\) was varied from 0 to 1. \(h/L = 1\) represent the case of pile embedded in single layer of clay only, whereas \(h/L = 0\) represent the case of pile embedded only in dense sand layer. Also, value of \(L/D\) was varied from 11.8 to 60 for soil strata with a constant value of clay layer thickness of 3m to study the effect of pile embedment in lower dense sand layer on the lateral load capacity of pile. Details of other parameters used in the study are given in Table 4. Figures 3(a) - 3(c) show typical Plaxis output showing deformed mesh, total displacements (\(u_x\)) and pile deflection respectively at lateral displacement of 0.2D for group 3x1 with \(L/D = 30\) and \(h = 3\) m.

Table 1. Details of the parameters used in the numerical analysis

<table>
<thead>
<tr>
<th>Thickness of clay layer, (h) (m)</th>
<th>Pile Length, (L) (m)</th>
<th>(L/D)</th>
<th>(h/L)</th>
<th>Number of piles, (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0, 1, 1.5, 2, 2.5, 3, 3.81</td>
<td>3.81</td>
<td>15</td>
<td>0, 0.26, 0.39, 0.52, 0.66, 0.79, 1</td>
<td>1</td>
</tr>
<tr>
<td>0, 1, 2, 2.5, 3, 4, 5.08</td>
<td>5.08</td>
<td>20</td>
<td>0, 0.2, 0.39, 0.49, 0.59, 0.79, 1</td>
<td>1</td>
</tr>
<tr>
<td>0, 1, 1.5, 2, 3, 4, 5, 6.35</td>
<td>6.35</td>
<td>25</td>
<td>0, 0.16, 0.24, 0.31, 0.47, 0.63, 0.79, 1</td>
<td>1</td>
</tr>
<tr>
<td>0, 1, 2, 3, 4, 5, 6, 7.62</td>
<td>7.62</td>
<td>30</td>
<td>0, 0.13, 0.26, 0.39, 0.52, 0.66, 0.79, 1</td>
<td>1, 2, 3</td>
</tr>
<tr>
<td>3</td>
<td>3, 3.048, 3.429, 3.81, 5.08, 6.35, 7.62</td>
<td>11.8, 12, 13.5, 15, 20, 25, 30, 45, 60</td>
<td>1, 0.98, 0.87, 0.79, 0.59, 0.47, 0.39, 0.26, 0.2</td>
<td>1</td>
</tr>
</tbody>
</table>
3. Validation of Finite Element Model

Before generation of prototype models, the adopted small finite element model was validated by comparing the results of finite element analysis to the laboratory model tests performed on single pile and group piles of patterns 2×1 and 3×1 embedded in soft clay overlying dense sand in the test setup reported by Kaur et al. (2021) [23]. The laboratory model tests were performed on steel pipe piles of 381 mm length, 12.7 mm outer diameter and 0.4 mm wall thickness in a tank of square cross section of 2×2 m and 1 m depth. Thickness of clay layer and sand layer were 150 mm and 850 mm respectively. For numerical analysis, a finite element model of same size and similar pile dimensions and soil conditions as in laboratory model tests was created using Plaxis 3D. Comparison of laboratory test results and the results obtained from finite element modeling of single and group piles are shown in Figures 4(a) to 4(c). A marginal difference between the experimental and Plaxis curves indicated a good agreement between the results from laboratory tests and finite element analysis. The obtained comparative data showed relevancy of the adopted model for simulating the pile – soil interaction and analysis of behavior of pile foundations.

Figure 4. Comparison of finite element analysis results with experimental test results for (a) Single Pile; (b) Group 2×1 and (c) Group 3×1
4. Results and Discussion

4.1. Effect of Clay Layer Thickness on Lateral Capacity

Figure 5 shows the lateral displacement -lateral load curves obtained for single piles with L/D ratio 30 for different values of clay layer thickness. Trends of the curves indicate a decrease in lateral capacity of the piles with increase in the thickness of clay layer. Similar trends were observed for group piles also. This variation was studied by comparing change in lateral load corresponding to lateral displacement of 0.2D to the variation in h/L for pile length 7.62m as shown in Figure 6. Successive reduction (%) in lateral capacity for all the pile group arrangements with change in h/L ratio is shown in Figure 7. It is visible from Figures 6 and 7 that for single as well as group piles, rate of reduction in lateral capacity with increase in clay layer thickness is higher at smaller values of h/L. The maximum reduction is observed for h/L varying from 0.26 to 0.39. The rate of reduction diminishes as value of h/L exceeds 0.39. Value of h/L of 0.39 corresponds to clay layer thickness of 3 m and depth of pile embedment in dense sand (H) equal to 4.62 m which is about 18 times pile diameter i.e. H/D = 18. The major reduction in lateral capacity was observed when depth of pile embedment in dense sand reduced from 30D to 18D for the considered pile length of 7.62 m with L/D = 30.

Figure 5. Lateral displacement versus lateral load curves for single pile with L = 7.62m, L/D = 30

Figure 6. Variation in lateral capacity with change in h/L for L = 7.62m, L/D = 30
Figure 7. Successive reduction in lateral capacity with variation in h/L.

Figure 8 shows that at h/L = 0.39, there is a reduction of order of 66.2, 69.1 and 70% in lateral capacity of single, group 2×1 and group 3×1 piles respectively with respect to the lateral capacity of piles in dense sand (h/L = 0). For piles embedded in soft clay (h/L = 1), lateral capacity of single, group 2×1 and group 3×1 piles reduces to 78.4, 85 and 85.5% respectively in comparison to piles embedded in dense sand (h/L = 0). Figure 8 clearly indicates that percentage reduction in lateral capacity in layered soil with respect to dense sand increases with increase in number of piles and is the largest for group 3×1 piles. However, it is also noticeable that group effect on variation in lateral capacity diminishes with the increase in clay layer thickness.

Figure 8. Reduction in lateral capacity in layered soil in comparison to dense sand

Change in lateral capacity was studied in terms of a non-dimensional parameter $F_x$ Ratio which relate the lateral capacity of piles in layered soil ($h > 0$) to that in dense sand ($h = 0$) and is defined as per Equation (1). Variation in $F_x$ Ratio with change in h/L for single pile with different values of L/D ratio is shown in Figure 9 which shows that $F_x$ Ratio decreases non-linearly with increase in h/L.

$$F_x\text{ Ratio} = \frac{F_x \text{ at lateral displacement } 0.2D \text{ in layered soil}}{F_x \text{ at lateral displacement } 0.2D \text{ in dense sand}}$$
Observed results of the numerical analysis showed that the lateral capacity of group piles and single pile embedded in soft clay (h/L = 1) ranges from 0.145 to 0.216 times the lateral capacity in dense sand (h/L = 0). It was observed that for all the pile group arrangements, $F_x$ Ratio reduces exponentially with increase in h/L. Nonlinear generalized reduced gradient approach was used to predict relationship between the two parameters which is applicable for single as well as group piles as given in Equation 2. Sum of squared residuals for the predicted relationship was 0.095 which indicated a good match between observed and predicted data. However, to apply correction for number of piles in the group (n), Equation 2 was revised to Equation 3 which produced a better match with smaller value of sum of squared residual of 0.075. Observed variation in $F_x$ Ratio with change in h/L ratio for single as well as group piles and the trends of predicted equations are presented in Figure 10. It is noticeable from the figure that the predicted expressions reasonably match with the observed trends.

$$F_x \text{ Ratio} = 1.0015 \ e^{-2.413 \ \frac{h}{L}}$$

$$F_x \text{ Ratio} = \left(1.0728 \ e^{-2.389 \ \frac{h}{L}}\right) \times n^{-0.1226}$$

Observed trends of variation of lateral capacity with change in h/L ratio fairly agree with the observations reported by Uncuoğlu and Laman [30] that lateral load capacity of piles in the layered sand conditions decreases non-linearly as the thickness of the upper layer increases. Similar observations were also suggested by Kim and Kim [31] that the effect of height ratio of non-homogeneous soil on deflection is exponential function with height ratio. Furthermore, it is to be noted that the predicted expressions are applicable only for similar conditions of soil stiffness, pile geometry and pile end conditions as considered in the present analysis.
4.2. Effect of Clay Layer Thickness on Lateral Deflection and Bending Moment

Variation in deflected shape of single pile and group 2×1 with increase in clay layer thickness is shown in Figures 11(a) and 11(b) respectively. It can be seen from the figure that the lateral deflection of the pile at a particular depth increases with increase in the thickness of clay layer. It occurs due to reduction in lateral stiffness of the soil which causes reduced lateral capacity of the piles. Bending moment profiles of single and group piles embedded in dense sand (h = 0) and in soft clay (h = 7.62 m) are compared in Figures 12(a) to 12(c). The maximum bending moment in the piles embedded in soft clay is noted to be in the range of 20% to 26% of the maximum bending moment in piles embedded in dense sand. Figure 13 shows a clear variation in bending moment profile of single pile with different values of clay layer thickness. Similar trend is noted in group piles also. Variation of the maximum bending moment with change in h/L ratio for single and group piles is presented in Figures 14(a) to 14(c).

![Figure 11. Lateral deflection profile for (a) Single pile and (b) Group 2×1](image1)

![Figure 12. Comparison of bending moment profile in sand (h= 0) and clay (h= 7.62 m) for (a) Single pile, (b) Group 2×1 and (c) Group 3×1](image2)
Figure 13. Plaxis output for bending moment profile of single pile with \( L = 7.62 \) m for different values of \( h \)
It is observed that for all the three pile setups, maximum bending moment in an individual pile initially increases with increase in h/L ratio. The largest value of maximum bending moment in all the piles is obtained at h/L = 0.26, except for front pile of group 3×1, in which the largest value of maximum bending moment is obtained at h/L = 0.13 and this value decreases with further increase in h/L ratio. It is also observed that at smaller values of h/L, maximum bending moment occurring in single pile is larger than the group piles and its value reduces with increase in number of piles in the group. But for h/L more than 0.39, there is not much difference in value of maximum bending moment for single and group piles. It is also observable that the bending moment in front pile of a group is larger than that in the trailing piles for h/L less than 0.39 and this trend also disappear for h/L greater than 0.39. These observations clearly show that group effect on lateral resistance as well as bending moment reduces as thickness of soft clay layer increases and it becomes negligible as clay layer thickness exceeds 40% of pile length.

4.3. Effect of Pile Length Variation on Lateral Capacity and Bending Moment Profile of Single Pile

To study the effect of variation in pile length on lateral capacity of single pile in layered soil, pile length was varied while keeping a constant clay layer thickness of 3m. Minimum length of the pile used in the analysis was taken equal to the thickness of clay layer i.e. 3m with value of L/D =11.8. Change in lateral capacity with increase in pile length is presented through the lateral displacement versus lateral load curves as given in Figure 15. Trends of the curves clearly indicate a significant enhancement in lateral capacity with increase in length up to L/D equal to 15. However, for higher values of L/D ratio, there is only a negligible increase in lateral capacity.
These results fairly agree with the observations of Reese and Van Impe (2010) [32] who reported that the depth up to 10D is of predominantly importance in soil-pile interaction in case of lateral loading. Abdurabo and Gaaver [33] also suggested that the effective depth of a flexible laterally loaded pile embedded in cohesionless soil is about 16 times the pile diameter.

An attempt was made to understand the effect of pile length variation in terms of depth of pile embedment in dense sand layer, H. It was observed that lateral capacity increases initially with increase in pile length because of increased stiffness of dense sand along depth until the pile attains an optimum embedment depth in dense sand beyond which there is not any significant change in lateral capacity with further increase in pile embedment. Observed data in the present combination of layered soil with clay layer thickness of 3m, showed that significant improvement in lateral capacity occurs till the embedment depth of pile in dense sand reaches a depth equal to three times pile diameter i.e. H/D =3 as shown in Figure 16.

![Figure 15. Lateral displacement versus lateral load curves for single pile, h = 3 m](image)

The improvement in lateral capacity with increase in pile length was also studied by comparing the variation in the ratio of lateral capacity in layered soil to the lateral capacity in soft clay ($F_{x}/F_{x\text{-clay}}$) and the change in normalized depth of embedment in dense sand in terms of pile length i.e. H/L as shown in Figure 17. The obtained curve clearly shows that optimum embedment depth in dense sand is about 21% of pile length i.e. H/L = 0.21 corresponding to value of $F_{x}/F_{x\text{-clay}} = 2.6$, indicating about 160% increase in lateral capacity in layered soil as compared to the soft clay. Maximum value of $F_{x}/F_{x\text{-clay}}$ is 2.73 for H/L = 0.8, which clearly indicates a marginal improvement in lateral capacity with increase in H/L beyond 0.21. It shows that major improvement in lateral capacity is achieved when 21% of pile length is embedded in dense sand in the soil layer combination adopted in the present analysis.

![Figure 16. Variation in lateral capacity of single pile with change in H/D in layered soil with h = 3 m](image)

Bending moment profile of single pile for different values of L/D ratio in layered soil with clay layer thickness of 3m is shown in Figure 18. A significant increase in maximum bending moment with increase in pile length up to L/D 15 is noticeable from the observed results. Maximum bending moment observed in pile with L/D 11.8 was 14.99 kNm which increased to 65.24 kNm at L/D 15. However, a negligible increase in maximum bending moment was observed.
with further increase in pile length. It changed from 70.54 kNm for L/D 20 to 75.47 kNm for L/D 60. A small amount of negative bending moment was also observed in pile lengths with L/D equal to 25 and more.

Figure 17. Variation in $F_x/F_{x\text{-clay}}$ with change in H/L for $h = 3$ m

Figure 18. Bending moment profile of single pile with different L/D ratio values for $h = 3$ m
4.4. Scale Effect on Lateral Capacity

In addition to the parametric study conducted in the numerical analysis, an effort was also made to study the scale effect on lateral load capacity. The size of prototype finite element model was varied using variety of scales (s times the size of experimental model) while keeping soil properties and pile end conditions similar. Different values of scale (s) employed in the finite element analysis were 20, 30, 40, 50 and 60. Scale effect was measured by comparing the variation in non-dimensional factor lateral capacity ratio (LCR) with the variation in the scale. LCR is defined as given in Equation 4.

\[
LCR = \frac{F_{xp}}{F_{xm}}
\]  

(4)

Here, \(F_{xp}\) is lateral load capacity of finite element model of prototype size with scale, s and \(F_{xm}\) is lateral load capacity of small finite element model with size equal to experimental model. Figure 19 shows that values of LCR corresponding to lateral displacement 0.2D and 0.1D vary by power law as the scale of the prototype varies. The obtained equations indicated that the value of ratio of lateral capacity of a prototype to a small model is around 10 times the square of the scale used. \(R^2\) values for the predicted relationships as given in Equations 5 and 6 for lateral displacement 0.2D and 0.1D respectively are 0.999 and 0.997, which show a good match with the data of finite element analysis results.

\[
LCR \text{ at lateral displacement 0.2D} = 10.053 \, s^{2.03}
\]  

(5)

\[
LCR \text{ at lateral displacement 0.1D} = 9.983 \, s^{2.065}
\]  

(6)

5. Conclusions

A three dimensional numerical analysis was performed using Plaxis 3D software to study the lateral capacity and bending behavior of single and group piles embedded in two-layered soil consisting of upper soft clay layer with water content equal to liquid limit overlying a dense sand layer with 80% relative density. Effects of variation in pile length and thickness of clay layer on lateral resistance and bending profile of the piles were studied.

It is concluded that increase in clay layer thickness has a detrimental effect on the lateral load resistance of piles. It causes larger decrease in lateral capacity of group piles. For piles embedded in soft clay in comparison to the piles in dense sand, the noted reduction in lateral capacity is 78.4, 85 and 85.5% in single, group 2×1 and group 3×1 piles respectively. Lateral capacity of piles in the layered soil decreases non-linearly with increase in the thickness of soft clay layer. \(F_x\) ratio reduces exponentially with increase in h/L ratio. Minimum value of \(F_x\) ratio is obtained for h/L =1 and it remains between 0.145 and 0.216 for group piles and single pile respectively. A generalized expression was developed for relationship between \(F_x\) ratio and h/L ratio which is applicable for single piles as well as group piles. The maximum bending moment in the piles embedded in soft clay at liquid limit is found to be in the range of 20 to 26% of the maximum bending moment in the piles embedded in dense sand. It is also concluded that the group effect on lateral resistance and bending moment in the piles reduces with increase in thickness of clay layer. There is a marginal difference in maximum bending moment in individual piles in a group and single pile in layered soil with clay layer thickness more than 40% of pile length. For a fixed value of clay layer thickness, lateral capacity and
maximum bending moment in single pile increases with increase in pile length up to an optimum embedment depth in dense sand. Further increase in the pile length beyond optimum embedment depth has marginal effect on lateral resistance of pile. The optimum embedment depth in dense sand is about three times pile diameter and 0.21 times pile length in layered soil combination considered in the present study. The optimum value of L/D ratio for single pile is 15 in the considered layered soil with clay layer thickness of 3m. Lateral capacity of pile embedded up to the optimum depth in layered soil is 2.6 times the lateral capacity of the pile embedded in soft clay. A generalized expression was developed for scale effect on the ratio of lateral capacity of prototype to that of model which is governed by a power law.

6. Declarations

6.1. Author Contributions

A.K., H.S. and J.N.J. contributed to the conception and design of the study; A.K. performed the experimental tests and numerical study and analysed the data; A.K. wrote the first draft of the manuscript; J.N.J. and H.S. guided and supervised the research work and commented on the previous version of the manuscript. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

6.4. Acknowledgements

The authors are thankful to I.K. Gujral Punjab Technical University, Jalandhar, Punjab, India and Civil Engineering department of Guru Nanak Dev Engineering College, Ludhiana, Punjab, India for the permission granted to use the research laboratories and Plaxis 3D software required in the study.

6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Primarily Results of a Real-Time Flash Flood Warning System in Vietnam

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Received 14 January 2021; Revised 11 March 2021; Accepted 19 March 2021; Published 01 April 2021

Abstract

In recent years, losses and damages from flash floods have been steadily increasing worldwide as well as in Vietnam, due to physical factors, human activities, especially under a changing climate. This is a hotspot issue which requires immediate response from scientists and policy-makers to monitor and mitigate the negative impacts of flash floods. This study presents a way to reduce losses through increasing the accuracy of real-time flash flood warning systems in Vietnam, a case study developed for Ha Giang province where the topography is relatively complex with severe flash floods observed. The objective of this paper is to generate the real-time flash flood system based on bankfull discharge threshold. To do this, HEC-HMS model is applied to calibrate and validate observer inflow to the reservoir with nine automatic rain gauges installed. More importantly, on the basic of measured discharge at 35 locations from the fieldtrips, an empirical equation constructed is to identify the bankful discharge values. It bases on the relationship between basin characteristics of river length, basin area and bankfull discharge. The results indicate an effective approach to determine bankfull threshold with the established-empirical equation. On the scale of a small basin, it depicts the consistency of flood status and warning time with the reality.

Keywords: Flash Flood; FFG; Real-Time; Bankfull Discharge, Ha Giang Province.

1. Introduction

In recent years, flash flood events have caused particularly serious consequences in Ha Giang, a province in the northwest of Vietnam. It has become more complex and more extreme in frequency of occurrence and intensity. In the period of 2012 to 2016, more specifically, Ha Giang recorded 15 flash flood events killing 70 people, injuring 82 peoples, and damaging 19500 houses and many crop areas. The damage cost was estimated at 1500 billion VND [1]. The extreme flash flood events which can be listed are one killing 7 people in July 2014 in Hoang Xu Phi district, and one sweeping 3 others away in September 2015 [2]. Currently, mitigation measurements of this disaster are facing many challenges due to a complex interaction mechanism of natural conditions. They are meteorology, geology, topography, and climate change. In addition, flash floods often occur at the same time with other natural disasters, such as landslides, making research even more difficult. Consequently, the asset and human losses derived from the flash floods are severely recorded in Ha Giang province where its topography is fairly complex with steep slopes. Although there have been many studies on flash flood, knowledge about its mechanism is limited. Flash floods have different features from floods in river, notably short time lag and occur in small mountain catchments with few hundred square kilometers or less [3]. This shows that forecasting of flash floods is quite challenging compared to...
traditional flood forecasting approaches. According to Miao et al. [4], the monitoring system in areas where flash floods occur is often insufficient. This is exactly the case for a sparse station density like Ha Giang area. There are only 33 hourly rainfall stations on 7945.8 square kilometers of the province (http://kttvqg.gov.vn).

Dealing with flash floods, there has been much attention from scientists for several decades [5-7]. Kumar et al. [8] implemented a comparison of Emotional Neural Network with Artificial Neural Network for modeling rainfall-runoff in the Sone Command, Bihar where flood events were frequently recorded under conditions of heavy rainfall. Using a high resolution model of distributed rainfall-runoff, Takahiro et al. [9] also investigated ensemble flash flood predictions in cases of heavy rain events in Japan. It showed the different results for each heavy rain events. In general, there are lots of approaches to study the flash flood. Water amount, an indicator, is importantly used to identify flash flood in the river that exceeds the transport capacity of the river. The river discharge at this moment is called bankfull discharge. Many authors identified bankfull discharge values in the range from 1 to 2.5 years returned interval [10, 11]. This approach faces the difficulty because calculated basins are often ungauged catchments. Moreover, the channel geometry strongly affects the value of bankfull discharge. The bankfull discharge can also be identified by field survey. Bankfull stage, which is water level corresponding with bankfull discharge, can be recognized in the field by several physical indicators and characteristics along the stream’s banks. These indicators discussed in Carpenter et al. [6] and Mulvihill [12] and proposed the regression equation between the bankfull discharge and the basin characteristics, which can be easily identified using geography information system technique [6, 12].

Bent et al. [13] developed this approach for streams in Massachusetts. This approach will enhance the accuracy of the important locations where channel geometry is surveyed. For location without conditional investigations, the regression equation will be used to reduce the amount of work. The next issue in flash flood warning process is to identify the rainfall threshold [4]. Flash Flood Guidance (FFG) method has been used in many studies [14-16]. FFG is the rainfall in a certain period (1-, 3- or 6-hour), causing water in the river to reach bankfull discharge value. The value of FFG is temporal variation and is closely related to soil moisture. According to Borga et al. [3], the impact of antecedent soil moisture on flow is nonlinear and important. Georgakakos [14] used Sacramento soil moisture accounting (SAC) model to describe the complex process of soil moisture. The SCS-Curve Number (SCS-CN) method [17, 18] is also a common method to define soil moisture condition which were discussed in researches [3, 19].

Flash flood forecast is specially a complex issue due to depending on many factors (e.g., meteorological conditions or river characteristics). As illustrated in lots of studies that meteorological contributions to flash flood are significantly important [20-22]. The reason for this is that extreme rainfall events potentially leading to pluvial flash floods closely associates to air moisture like dewpoint temperature, relative humidity or precipitable water. However, one of the most important factors is the river characteristics (e.g., river cross section, river slope) [23-24]. They identify the transport capacity river segments and decide the bankfull discharge threshold. Another important contributing factor of flash flood event is the current basin condition. Basin conditions can be expressed through soil moisture as well as current river water levels. This condition will determine the FFG value at each time. The accuracy of forecast and nowcasting rainfall also plays a decisive role for flash flood warning. In this study, the authors mentioned all these factors to improve the accuracy of the flash flood warning system via a real-time flash flood warning system. This research process combines (1) designing the field survey to investigate bankfull discharge values; (2) developing the rainfall-runoff model associated with observer real time rainfall data to assess the current status of the basin; and (3) determining the FFG value in real time, combined with the forecasting rainfall to appropriate flash flood warnings. The study is applied to the Nam Ly and Na Nhung basins in Ha Giang province, Vietnam.

2. Materials and Methodology
2.1. Research Area

This study was conducted for two catchments of Nam Ly and Na Nhung River in Ha Giang province, Vietnam. Areas of the two basins are 92.72 and 41.46 km², respectively. These basins have a steep slope of 44.17% for Nam Ly and 58.82% for Na Nhung. The study basins are located near the rainfall band eye Bac Quang with an annual rainfall of about 4000 mm. In these areas, flash floods often occur with serious consequences. Figure 1 shows the location of the study basins.
2.2. Overview of Methods

The research method is presented in the Figure 2. Based on the field survey data, the values of the bankfull discharge at the survey locations are determined. A regression equation is developed between the basin characteristics and bankfull discharge value. This equation is used to calculate bankfull discharge values for all sub-basins. The semi-distribution models are conducted for the research basins. The research used real-time rainfall data from the station system in the basins as input to the mathematical model. The peak discharge at the outlet of the basin would be continuously updated and compared with bankfull discharge to define the FFG values. In this study, the forecasting rainfall product from the Global Environmental Multiscale Model (GEM) model is selected [25]. The forecast rainfall
value is compared with the FFG value in each time to issue the appropriate warning message. The flowchart of computing processes is described in these sections below.

2.3. Bankfull Discharge

Research did a survey at 35 locations in two basins of Nam Ly and Na Nhung (Figure 1). The investigated locations are shown in Figure 1. The selected locations included main streams and tributaries to increase the representativeness of collected data. In each location, channel geometry is measured. In addition, the discharge value at 2 times in the flood and dry seasons are determined. Based on these data collected, the discharge – water level curve \( Q = f(H) \) for each location used the Manning’s Equation 1:

\[
Q = \frac{1}{n} A R^{2/3} S^{1/2}
\]

Where: \( Q \): Discharge (m³/s); \( A \): Flow Area (m²); \( R \): hydraulic radius (m); \( S \): Slope of Energy Gradient (-); \( n \): Manning’s Roughness Coefficient

In the right-hand side of the equation, \( A \) and \( R \) are determined from cross section data corresponding with each water level. The \( n \) value of each cross section is preliminary selected following instruction of Barnes (1967) [26] depending on investigated locations. The value of \( S \) can be assumed to equal river slope at these locations. The \( Q = f(H) \) curve should be adjusted to fit observer discharge values.

At each cross-sectional measurement site, the bankfull indicators are identified. The Figure 3 presents an example of cross section at Nam Choong village, Quang Nguyen commune. In this location, changes in slope from vertical bank to a horizontal flood plain are the identifier bankfull indicators. The detail archived data is listed in the Table 1.

Based on the \( Q~H \) curves which mentioned above, bankfull discharges at these locations are calculated.

![CROSS SECTION Q9](image)

**Figure 3. An example of cross section Q9 in Nam Ly basin**

Based on the topographic map at a scale of 1: 10000, the Nam Ly and Na Nhung basins were divided into 38 and 27 sub-basins, respectively (Figure 1). The area of each sub-basin varies from 0.7 to 4.5 km². The outlets of these sub-basins are the junction or where populations are close to the river. The watershed division considers the spatial distribution of rainfall which causes the difference in the flow on the tributaries. At each outlet of sub-basin, discharge should be carried out via mathematical model. Therefore, flash flood warning can be done with more detailed at these locations.

Due to data limitations, measurements could not be made at all sections, so an alternative method was used to estimate bankfull discharge for whole basins. The survey locations are presented in Figure 1 and corresponding sub-basin characteristics is shown in table 1. In this table, \( F \) is basin area, \( L \) is mainstream length, \( Z \) is average elevation of sub-basin, \( S_{river} \) and \( S_{basin} \) is river slope and basin slope, respectively, \( CN \) is CN index which was estimate by land cover data. A regression equation between the bankfull discharge and the basin characteristics is established based on survey data. This equation was used to define bankfull discharge values for all sub-basins.
Table 1. Bankfull stage and bankfull discharge value at investigated location

<table>
<thead>
<tr>
<th>ID</th>
<th>Name</th>
<th>Bankfull stage (m)</th>
<th>Bankfull discharge (m³/s)</th>
<th>F (km²)</th>
<th>L (km)</th>
<th>S_{base} (%)</th>
<th>Z (m)</th>
<th>S_{river}</th>
<th>CN</th>
</tr>
</thead>
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<td>1</td>
<td>Q1</td>
<td>817.44</td>
<td>41.55</td>
<td>6.63</td>
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<td>0.737</td>
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<td>0.276</td>
<td>67.714</td>
</tr>
<tr>
<td>2</td>
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<td>653.30</td>
<td>27.78</td>
<td>1.73</td>
<td>1.875</td>
<td>0.773</td>
<td>835.75</td>
<td>0.265</td>
<td>65.412</td>
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<tr>
<td>3</td>
<td>Q3</td>
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<td>78.66</td>
<td>18.94</td>
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<td>0.167</td>
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<tr>
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<td>926.08</td>
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<td>0.222</td>
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<td>0.142</td>
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<td>72.63</td>
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<td>20</td>
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<td>0.273</td>
<td>61.345</td>
</tr>
</tbody>
</table>

Note: F and L is basin area and mainstream length up to basin outlet, they were defined from topography data.

2.4. Semi-Distribution Model

2.4.1. Realtime Rainfall System

To establish a flash flood warning system, the study required 9 automatic rainfall stations on 2 basins. 4 stations were setup in Nam Ly basin and 5 stations were in Na Nhung. Figure 1 and Table 2 present the location of rainfall station system. The observation rainfall data was updated continuously to the system with 5-minute interval. This data was input for rainfall runoff model to define real time condition of the basin.
Table 2. Location of rainfall station system

<table>
<thead>
<tr>
<th>ID</th>
<th>Station</th>
<th>Lat.</th>
<th>Long.</th>
<th>Commune</th>
</tr>
</thead>
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<td>1</td>
<td>QN01</td>
<td>22°34'29&quot;</td>
<td>104°33'18&quot;</td>
<td>Quang Nguyen</td>
</tr>
<tr>
<td>2</td>
<td>QN02</td>
<td>22°33'41&quot;</td>
<td>104°31'55&quot;</td>
<td>Quang Nguyen</td>
</tr>
<tr>
<td>3</td>
<td>QN03</td>
<td>22°32'49&quot;</td>
<td>104°33'22&quot;</td>
<td>Quang Nguyen</td>
</tr>
<tr>
<td>4</td>
<td>QN04</td>
<td>22°36'52&quot;</td>
<td>104°34'42&quot;</td>
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</tr>
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<td>5</td>
<td>TSC01</td>
<td>22°40'49&quot;</td>
<td>104°44'20&quot;</td>
<td>Ta Su Choong</td>
</tr>
<tr>
<td>6</td>
<td>TSC02</td>
<td>22°41'44&quot;</td>
<td>104°45'30&quot;</td>
<td>Ta Su Choong</td>
</tr>
<tr>
<td>7</td>
<td>BN01</td>
<td>22°42'41&quot;</td>
<td>104°45'31&quot;</td>
<td>Ban Nhung</td>
</tr>
<tr>
<td>8</td>
<td>BN02</td>
<td>22°42'70&quot;</td>
<td>104°44'43&quot;</td>
<td>Ban Nhung</td>
</tr>
<tr>
<td>9</td>
<td>BN03</td>
<td>22°42'60&quot;</td>
<td>104°44'33&quot;</td>
<td>Ban Nhung</td>
</tr>
</tbody>
</table>

2.4.2. Semi-distribution Model

The processes of calibration and validation were conducted in Nam Ly basin 1. The area of this basin is 76.4 km². Nam Ly basin 1 is a part of Nam Ly basin. The outlet of Nam Ly 1 basin has a hydropower construction, which measures inflow to the reservoir. This construction has just operated thus only data of 2 flood events at June 4, 2018 and June 24, 2018 were observed. The calibration process and process validation were carried out with data collected from these events.

Based on the results of the sub-basin division, the study established the HEC HMS semi-distribution models for Nam Ly and Na Nhung basins. The structure of semi distributed models can be found in Figure 4. The similar model framework was described in Zhai et al. [27]. In this model, each sub-basin was described by an independent component. The transformation process from rainfall to discharge was carried out through 3 components: loss, transform, and baseflow. Loss is carried out continuously in both dry and wet conditions. The model simulated in both dry and wet conditions, so the SMA model was used. Details of SMA were elaborated in Bennett et al. [28]. The transformation process from excess rainfall to direct runoff via Soil Conservation Service (SCS) dimensionless unit hydrograph [18]. According to Mishra et al. [17] publication, this method was simple and useful for ungauged watersheds. Baseflow was sustained runoff of prior precipitation that was stored temporarily in the watershed [17]. This flow plays an importance role in dry condition. The exponential recession model [29] has been used to present the recession of flow. This is made to ensure that an almost zero flow can occur in the sub basins after a long period without rain. In this model, sub basins were linked by river segments. Muskingum method had been selected to rout water in river segments. The structure of models is shown in the Figure 4.

![Figure 4. Hec-HMS model for Nam Ly (a) and Na Nhung (b)](image-url)
2.5. Forecasting Rainfall and Flash Flood Guidance

2.5.1. Forecasting Rainfall

Forecasting rainfall data is adopted from the Global Environmental Multiscale Model (GEM) [18]. This model, often known as the CMC model in North America, is an integrated forecasting and data assimilation system developed in the Recherche en Prévision Numérique (RPN), Meteorological Research Branch (MRB), and the Canadian Meteorological Centre (CMC). GEM is a non-hydrostatic atmospheric model and described briefly in Cote et al. (1998). The physical parameteristics mainly included (1) Planetary boundary layer based on turbulent kinetic energy; (2) fully-implicit vertical diffusion; (3) stratified surface layer, distinct roughness lengths for momentum and heat/moisture; (4) improved force-restore method for land surface processes (evapotranspiration, snowmelt, soil types); (5) solar/infrared radiation schemes with cloud-radiation interactions based on predicted cloud radiative properties; (6) shallow convection parametrization; (7) Fritsch-Chappell deep convection mesoscale scheme with diagnostic cloud properties; (8) explicit cloud water/ice prediction scheme (Sundqvist) with quasimonotone semi-Lagrangian 3D advection and (9) gravity wave drag parametrization. The model is run two times a day and produces forecasts for up to 240 hours with the interval of 3 hours. In this study, the lead-time up to 24 hours with a resolution of 15 kilometers is considered for testing flash flood warning system. Figure 5 shows values of rainfall at grid cells from the model.

2.3.2. Performance of GEM Weather Forecast Model

As first step, skills of the GEM run are compared against the rain gauge in time and space. The spatial comparison is done for 8 grid cells. It is emphasized that the gridded precipitation is created for the whole Ha Giang province. The data from automatic rain gauges during three episodes of June 1-14, 2020; July 04 - 07, 2020 and July 19 - 22, 2020 is used to confirm the performance of GEM run. The temporal comparison uses hourly accumulations for the stations located in the basin. The following metrics are used for the degree of agreement between model and observed rainfall (i) the bias (Equation 3), (ii) the root-mean-square error (RMSE; Equation 4) and (iii) the correlation coefficient $r$ (Equation 5):

\[
\text{Bias} = \frac{1}{N} \sum_{i=1}^{N} (M_i - O_i) \tag{3}
\]

\[
\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{N} (M_i - O_i)^2}{N}} \tag{4}
\]
\[ r = \frac{\text{Cov}(O_i - M_i)}{\text{SD}_i \text{SD}_i} \]  

(5)

Where \( M_i \) is the i-th value estimated from the model, \( O_i \) is the value observed at the terrestrial station and \( N \) is the number of data analysed, \( r \) is the linear correlation coefficient between \( O_i \) and \( M_i \); and \( \text{SM}_i \) and \( \text{SO}_i \) are the standard deviations of \( M_i \) and \( O_i \), respectively.

Importantly, HSS index [30] is applied and calculated using the Equation 6.

\[ \text{HSS} = \frac{(\text{hits} + \text{correct negatives}) - (\text{expected correct})_{\text{random}}}{N - (\text{expected correct})_{\text{random}}} \]  

(6)

Where  

\[ (\text{expected correct})_{\text{random}} = \frac{1}{N} [(\text{hits} + \text{misses})(\text{hits} + \text{false alarms}) + (\text{correct negatives} + \text{misses})(\text{correct negatives} + \text{false alarms})]. \]

The HSS values can range from -1 to 1, where the value of 1 indicates a perfect forecast; zero indicates no predictive ability or a forecast equivalent to a reference forecast.

2.5.3. Flash Flood Guidance

The principle of FFG definition is illustrated by Figure 6. In this figure, the amount of rainfall up to current situation is shown as back columns. The flow at the outlet of the basin caused by this amount of precipitation is indicated by the black line. The maximum discharge is compared to the horizontal black thin line as bankfull discharge value. In many cases, the maximum discharge does not reach the threshold value. The precipitation in the (pattern column) that causes the flow at the outlet is shown as a dash line. The accumulative flow at the outlet is plotted by a dot line. Through the trial and error process, an amount of rainfall will be determined so that maximum of dot line will reaches the threshold value which is shown in Figure 6. This rainfall value will be the FFG value 1h. This process is carried out continuously to determine the temporal basin condition as well as FFG value. For each time step, the value of FFG is compared with the forecasting rainfall value so that it can issue an appropriate warning.

![Figure 6. Methodology to define 1h FFG](image)

3. Results and Discussions

3.1. Regression Equations for Estimating Bankfull Discharge

The most accurate way to determining the basin characteristics is survey. However, the survey could not be conducted in the entire basin due to many factors. For that reason, research has developed regression equation to identify bankfull discharge from basin characteristics. The development of this regression equation is often developed by the authors based on exponential form. This has been mentioned in the research of Carpenter et al. [6]. In this study, the same method was also used.

An issue of concern is the selection of characteristics for the regression equation. Lumia et al. [31] Selected 14 variables to determine flood peak flow with different frequencies. According to Bent and Waite [13], among factors, basin area plays an essential part. However, other factors such as main-channel slope and elevation also have certain effects [32]. Manmade structure such as dams, weir, and diversions can also affect bankfull discharge, but in the study basin, small hydroelectric reservoirs have no effective volume. Therefore, this effect might be neglected. In this study,
the authors also tested with several variables that can be easily collected such as catchment area, main river length, catchment slope, and forest area ratio. For the study area the result in Table 3 shows that, the bankfull discharge is closely related to the catchment area and the length of the main river with correlation index are 0.97 and 0.94, respectively. As a result, these characteristics were finally chosen for developing multiple regression equations in these areas.

<table>
<thead>
<tr>
<th></th>
<th>Qbf</th>
<th>Sbasin</th>
<th>Zmean</th>
<th>Sriver</th>
<th>CN</th>
<th>F</th>
<th>L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Qbf</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sbasin</td>
<td>0.131</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zmean</td>
<td>0.101</td>
<td>0.017</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sriver</td>
<td>-0.733</td>
<td>0.326</td>
<td>0.153</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CN</td>
<td>0.150</td>
<td>0.137</td>
<td>0.139</td>
<td>-0.163</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>0.973</td>
<td>0.148</td>
<td>0.068</td>
<td>0.703</td>
<td>0.141</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>0.942</td>
<td>0.087</td>
<td>0.023</td>
<td>0.741</td>
<td>0.179</td>
<td>0.946</td>
<td>1</td>
</tr>
</tbody>
</table>

The multiple regression equations for bankfull discharge with these two variables are presented in Equation 7:

\[ Q_{bf} = 17.661F^{0.424}L^{0.197} \]  

Where \( Q_{bf} \): Bankfull discharge (m\(^3\)/s); F: Basin area (km\(^2\)); L: Main river length (km).

These selected variables offer many advantages. For example, the values of these two variables can be easily determined from the topographic data. Currently, remote sensing technology is developing rapidly, available topographic data sources are freely available and easy to collect (e.g., Global Digital Elevation Model, Shuttle Radar Topography Mission). Besides, with GIS tools, the catchment area as well as the main river length has been determined with very high accuracy. Nearly the whole process is automated and everyone will get the same result with same input. The sensitivity analysis was also performed with 10% variation of input variables. The result shows that the bankfull discharge only vary within 6%.

This equation was developed by using base-10 log-transformed bankfull discharge and basin characteristic data (Table 1) for all 34 study sites. The standard techniques of multiple regression analyses were evaluated by reviewing the Adjusted R Square (coefficient of determination). The Adjusted R Square for bankfull discharge (0.95) was good. The Significance F value in the ANOVA (analysis of variance) result is close to 0. It means that, this regression equation is reliable, and it can be extrapolated to other sub-basins. Similar to other study conclusions [31, 32], the basin area is a vital factor because its P value is the smallest.

As mentioned above, the survey will give appropriated value of bankfull discharge with each river segments. For this approach, the priority sites such as riparian areas will be selected in advance for the survey. Thereby, improving accuracy at these positions needs more attention. Using regression equations should minimize the survey work. The basin characteristics can also be easily determined from available data. However, in some cases, bankfull indicators may not easy to identify. In addition, the application of regression equations for basin out of survey data range might also bring some errors. To reduce error incidence, it should be considered when selecting measurement locations.

3.2. Selection of Forecasting Rainfall

Performance of the GEM model is examined using the continuous and dichotomous indices. These values are calculated from the basin-averaged rainfall values of GEM model as plotted from the Figure 5 and 04 and 05 automatic rainfall stations for basins of Nam Ly and Na Nhung, respectively. The results for 3 periods from June 1-14, 2020; July 04 - 07, 2020 and July 19 - 22, 2020 shows that the GEM model obtains the values for RMSE (12 mm/3 hours), Bias (0.7 mm/3 hours), r (0.12) and HSS (0.51) for Nam Ly basin. For Na Nhung basin, the values of Bias (-3.5 mm/3 hours), RMSE (13.5 mm/3 hours), r (0.22) and HSS (0.43) are calculated. The comparison with recorded rainfall, the GEM performance is specially emphasized and presented for the periods of 5-8 July, 2020 and 19-22 July, 2020. The reason for this is that the recorded rainfall from automatic stations sharply alters during these periods.

Figure 7 shows a distribution of daily rainfall between GEM model and recorded rainfall over Nam Ly and Na Nhung basins. Generally, it can be seen from this figure that GEM model unsuccessfully captures rainfall events that sharply change from day to day for Nam Ly. Contrary to this, GEM model could relatively capture the mount of rainfall over Na Nhung.
During the period of 19-22 July, 2020, it is specially pay attention to the GEM model product that produces the rain events in all spans of rain events for Nam Ly basin. Basically, the cumulative distribution function of GEM model well matches the distribution of measured rainfall as shown in Figure 7. Thus, it is deemed to be reliable and highly potential for hydrological and environmental applications at a temporal scale of 3 hours. It is noted that the model could not catch up the events of very extreme heavy rainfall recorded in Na Nhung basin (Figure 8). Regarding to this is very likely from the local types of rainfall due to an ensemble of complex mountain shapes and convective clouds.

Above analysis of model performance confirm that GEM products potentially use for hydrologic applications and thus they are selected for testing a real-time flash flood warning system developed for Ha Giang Province located in the northern of Vietnam in this study.

3.3. Flash Flood Warning System

The study area locates in Ha Giang province, Vietnam. This is a mountainous area with a steep slope, and it has many waterfalls along rivers. In this region, some segment rivers only have water during flood season. It shows that, hydraulic models can be unstable when being simulated. Considering updated rainfall interval is only lats 1 up to 10 minutes. As a result, all the process in warning system must be completed during this time. This leads that, the semi-distributed HMS distribution model is appropriated selection. This selection ensures the detail level as well as the reasonable simulation time run.

The major issue when using a mathematical model is to ensure its accuracy. In this study, the authors enjoy an advantage when collecting data to validate the model. However, in many cases, the insufficient data is very common.
It is thus necessary to take measures to preliminarily define the model parameters. Chau [33] developed a method to estimate main parameters for HMS model based on basin characteristics (e.g., land use, river length, river slope). This method was successfully applied for Vu Gia - Thu Bon river system. Duc et al. [34] were also successfully adopted by the same approach for Kone - Ha Thanh river basins. Based on this approach, parameters of NamLy 1 were found in this study. The correlation equations are constructed between the basin characteristics and validated model parameters. Using these equations to find the appropriate parameters corresponding to the characteristics of the calculated basin (Nam Ly & Na Nhung). The result of comparison between calculated and observer inflow to the reservoir is shows in figure 6. The Nash indexes are 66% and 67% for calibration and validation, respectively. These values indicate that the performance rating of model is good [35]. As is indicated by this figure, the peak discharge was well simulated in terms of the values and the time occurrence. These are also the most important issues to identify the flash floods. It leads that the study uses this set of parameters to simulate the study basins.

![Figure 9. Calibration (a) and validation (b) process of Nam Ly 1 basin](image)

After determining the set of parameters, distribution models were used to forecast flash floods. The input of the model is a combination of observer rainfall data and forecast rainfall from GEM product. The output of the model will be the discharge at the outlet of the sub-basin. With semi-distributed model, the system will take advantage of the basin's spatial rainfall station system. The spatial distribution of precipitation in the basin is very large, this will be demonstrated in the experimental prediction process part. Therefore, the model is capable of simulating the changes in the current state for each small sub-domain. Updating the current condition of the basin is particularly important, cause it define the FFG value at certain time and particular location. Compared to other studies in this region [36-37], that only showed flash flood susceptibility map. Our results go beyond previous reports, showing that flash flood can spatial and temporal forecast in our system.

The flexibility is also one of the system's advantages. In the future, under socio-economic development, some new residential areas may be formed, or some new important locations need to be further forecasted. The redistribution of sub-basins, as well as recalculation of sub-basin characteristics, can also be easily done. Therefore, the system is fully capable of changing to adapt to each specific requirement.

### 3.3. Experimental Forecast

The study carried out the experimental forecast in the research areas in 2020. In early flood season 2020, there were 3 extremely flood events happen in this region. The temporal rainfall distribution at all the station in the area is presented in Figure 10. The first event occurred on June 13-14th. This was an extremely rainfall in Nam Ly basin with rainfall amount up to nearly 215 mm (e.g., QN02). However, the total rainfall in Na Nhung ranged from 43 to 75 mm in this event. The rainfall caused severe flooding in Quang Nguyen town (Nam Ly basin) as Figure 11. Flooding did not occur in the Na Nhung basin. The second rain appeared on July 6th. The rainfall concentrated in the Nam Ly basin. Meanwhile, amount rainfall in Na Nhung was not too much. In this event, the massive rainfall intensity reached to 49 mm/h at QN01 station or 41 mm/h at QN03 station. It leaded to flooding in Quang Nguyen town (Nam Ly basin) as shown in Figure 12. The 3rd flood appeared in on July 20-21st. Heavy rains mainly occurred in Na Nhung basin. This was a flood causing the heavy damage to people and properties in the study area. Figure 12 shows some images at Coc Nam village, Ban Nhung commune (Na Nhung basin) after the flood.
Figure 9. The temporal rainfall distribution in 3 events

Figure 10. The flooding in Quang Nguyen town (Nam Ly basin) during the flood on 13-14 June 2020

Figure 11. The flooding in Quang Nguyen town (Nam Ly basin) during the flood on 06 July 2020
Figure 12. The picture in Coc Nam village, Ban Nhung commune (Na Nhung basin) after the flood on 21 July 2020

Table 4 shows the experimental forecast results of system. Due to the limitation of hydrology station in research area so it is difficult to define exact moment when water level gets over the riverbanks. The data used to verify was collected immediately after the event from local staffs. Therefore, this comparison is only approximate. However, the positive results have shown the potential capability of the system.

Table 4. Rainfall data in extreme event on June 13-14th

<table>
<thead>
<tr>
<th>Date</th>
<th>Nam Ly Basin</th>
<th>Na Nhung Basin</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Actual Situation</td>
<td>System Forecast</td>
</tr>
<tr>
<td>13-14 Jun</td>
<td>At 03:00 on 14th, the Quang Nguyen town (NL23) was inundated.</td>
<td>The warning message at sub basin NL23 (Quang Nguyen town) was released at 23h00 13rd.</td>
</tr>
<tr>
<td>05-06 Jul</td>
<td>At 4h00, Quang Nguyen town (NL23) was inundated.</td>
<td>The warning message at sub basin NL23 (Quang Nguyen town) was released at 01h15 06th.</td>
</tr>
<tr>
<td>20-21 Jul</td>
<td>Light rainfall in this area provided no indication of flooding.</td>
<td>No warning was announced by the system.</td>
</tr>
</tbody>
</table>

According to the measurement data, it is noticed that the complex spatial distribution precipitation across the research area. There was a big difference between amounts of rainfall on two basins during 3 calculated events, although the distance between these two basins was only 23 km. This shows that the essentials for installing rainfall system for each specific area. Based on these rainfall stations, the local rainfall was accurately measured. This contributes to improving the quality of the warning system in our study. Besides, through the survey process, the bankfull values (discharge and stage) in important locations are determined in a specific way, especially in residential areas when the river channel is strongly affected by artificial constructions. This value will help the system minimize errors compared to using the design flood value of P = 2% like other systems [4]. The results in table 4 confirm that this a good system for flash flood forecasting. The system has issued the correct warnings for all 3 heavy rain events in this region. From the results, it is clear that warning message was announced 3 to 4 hours before the moment when
Quang Yen town was recorded as inundation. Although, the starting point of inundation should be earlier. However, this area is small and flat. We speculate that the flooding process takes time cannot up to 3 hours. The results confirm that the system can provide early warnings. This will be very helpful for local residents as well as decision makers.

Although the system brings many key advantages, it also has limitations. After an operation period, the leaves might fall into the rain gauges, which affect the measurement results. To solve this problem, regular maintenance of rain gauges is required. This is also the reason why we set up these stations in residential areas for the study. Another limitation is the data transmission process performed through the cell signal. Because the systems are set up in the mountain areas, the interruption of cell signal may occur. It is very dangerous to lose the signal at the right time of heavy rain. This can only be overcome by improving the quality of the beacon. However, this is beyond the capabilities of our research.

4. Conclusion

The study has setup a real-time flash flood warning system for the Nam Ly and Na Nhung basins in Ha Giang province, Vietnam. The system is a combination of a number of factors to improve the accuracy of flash flood warnings. Firstly, a real-time rainfall system has installed in the research basin to measure spatial and temporal rainfall distribution. For areas with complex rain distribution such as the study area, this observer system is a prerequisite. Based on the comparison of observer and calculated rainfall, the study’s result shows that, the forecast rainfall GEM model has appropriate results with the BIAS indexes are 0.7 mm/3 hours and -3.5 mm/3 hours for Nam Ly and Na Nhung respectively. Another factor that contributes to improve the accuracy of our system is the approach to determine bankfull threshold. An empirical equation is established for the study area to determine bankfull discharge from basin characteristics. The empirical equation has a high relation coefficient (R² = 0.95). The system was experimental forecasted for the June three flood events in June and July 2020 for both basins Nam Ly and Na Nhung. The primarily results show that the status of the flood as well as the warning time of the system are consistent with the reality in the basins. However, it is necessary to carry out further performance for other flood events to confirm the reliability of the system.

5. Declarations

5.1. Author Contributions

Conceptualization, Tran Kim Chau and Nguyen Tien Thanh.; Methodology, Tran Kim Chau and Nguyen Tien Thanh.; Formal analysis, Investigation and data pre- and post-processing, Tran Kim Chau, Nguyen Tien Thanh, Nguyen The Toan.; Writing-original draft preparation, Tran Kim Chau.; Writing-review and editing, Tran Kim Chau, Nguyen Tien Thanh, Nguyen The Toan. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

Data sharing is not applicable to this article.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


A Review and Comprehensive Analysis of the Performance of University – Construction Industry Collaboration

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Received 13 January 2021; Revised 22 March 2021; Accepted 30 March 2021; Published 01 April 2021

Abstract

University–construction industry collaboration (UIC) has become an essential part of driving innovation and fostering construction industry growth. Measuring the performance of such collaboration is an emergent field of study that is scattered through the current literature. This study aims to identify the UIC performance indicators advocated by the top cited references, and map UIC performance indicators in the context of the state of Qatar. The present research evaluated the literature related to measuring the performance of UIC, considering publications in selected scientific databases over the period of 2004 – 2020. The publications were obtained through a search of the Science Direct, Emerald Insight, Scopus, Web of Science, Springer Link, SAGE, Research Gate, and Taylor & Francis Online databases. Keywords used in searching for publications included university, construction industry, business, cooperation, collaboration, relation, performance, and measurement. The findings were discussed and confirmed in the context of Qatar’s education and innovation ecosystem through semi-structured interviews with two renowned scholars who are involved in UIC. The study revealed that both universities and the construction industry are increasingly focused on measuring the performance of collaboration through specific performance indicators. The results show that both universities and the construction industry share some interests when it comes to four key performance indicators. These performance indicators are (1) the number of registered patents, (2) the number of patent applications, (3) the number of innovations (process/marketing/product/organisational), and (4) the number of publications. This study contributes to a general understanding of measuring UIC performance and defining trends in this research field. It also highlights research limitations and provides an arena for future research in the field.

Keywords: University; Construction; Industry; Business; Cooperation; Qatar; Collaboration; Relations.

1. Introduction

A key focus of transforming the construction industry is promoting a more innovative working environment through collaboration across the construction industry ecosystem. Such collaboration would have a vast effect on relationships between the construction industry and universities, among other partners [1]. Indeed, universities and the construction industry can cooperate in different ways. These include, but are not limited to, research and development, mobility of academics, mobility of students, commercialisation of research results, curriculum development, curriculum delivery, lifelong learning, spinoff and start-up formation, and university governance [2].

Likewise, types of university–construction industry cooperation that provides straight and measurable benefits have a tendency to be the most developed types of cooperation [3], such as research and development,
commercialisation, and student mobility. Thus, university–construction industry collaboration (UIC) can take several forms and practices through various activities. Commonly, such collaborations fall under one of three main activities: (1) collaborative training and education, (2) collaborative consulting and services, and (3) collaborative research [3-5].

Indeed, the outcome of such collaborations is realised differently by universities and the construction industry. From a university perspective, such outcomes are realised through aspects related to an enhanced learning environment, increased rates of knowledge creation, and better serving society and regional economies. From the construction industry perspective, such outcomes are realised by aspects related to improved innovation rates, revenue, and access to resources [6, 7].

Accordingly, UIC is essential to establish and nurture innovation ecosystems that drive the national innovation agenda and sustain economic growth [8]. In the United States, the most notable programme driving UIC is sponsored by the Bayh–Dole Act (1980) [9]. The comparable programme in Europe is the Horizon 2020 [10]. In Qatar, UIC is driven mainly by initiatives of the Qatar National Research Fund [11]. However, the economic dynamic between the two major actors of the innovation ecosystem faces a dilemma. It consists of two distinguished economies: the knowledge economy and the commercial economy. The first is driven by a need for advanced fundamental research or social value through universities and research centres, while the second is driven by the requirements of the marketplace of business entities [12]. This dichotomy makes measuring UIC performance a challenge for universities and the construction industry alike.

Indeed, there is a vast body of literature on the topic of UIC. However, these studies have focused on aspects related to technology transfer [13], UIC governance [14], collaboration forms or activities [15], commercialisation [16], university entrepreneurship [8], and UIC as open innovation [10]. The subject of measuring the performance of university–construction industry relations is scattered throughout the current literature, and this gap provided a basis for this study. In Qatar, there is a lack of objective performance management process of university-construction industry relations. Such a process is key to assessing UIC partnerships’ validity and credibility [Abduljawad, 2015]. Also, it shall provide the basis for the legitimacy of co-creation of value in the construction industry [17] and improving trust between UIC partners [18].

Therefore, this research sought to fill this gap and review most relevant literature on measuring the performance of UIC. We also conducted semi-structured interviews in order to refine, align, and interpret the findings of the literature review in the context of the state of Qatar. Therefore, this research, on the one hand, contributes to systematic literature reviews focused on UIC performance. On the other hand, it sheds light on context-related matters relating to UIC performance in Qatar. In this sense, this research presents a future research agenda for measuring UIC performance. This study aims to identify the UIC performance matrices advocated by the top-cited references and refine the performance matrices of UIC in the context of the state of Qatar.

2. Research Methodology

A systematic review was carried out on the topic of UIC. The search for published papers was carried out in Science Direct, Emerald Insight, Scopus, Web of Science, Springer Link, SAGE, Research Gate, and Taylor & Francis Online. We used keywords in searching for articles including university, construction, industry, university, business, cooperation, collaboration, relation, performance, and measurement. We selected papers for this review from the past 17 years (i.e., articles published since 2004). This restriction was chosen to reveal the most recent trends in this emergent field of research. The time period was also in line with the coverage limits of the Scopus database, as it is currently limited to articles published since 1995 [10].

The literature research was performed between 10 February 2018 and 31 December 2020. The literature research followed six steps, as displayed in Figure 1 below. In parallel, the researcher reached out to two active scholars in UIC from Qatar University. The researcher conducted semi-structured interviews with both scholars in order to reveal country-specific concerns, challenges, and priorities in regard to measuring UIC performance. The interviews were conducted on 19 February 2018 and 7 January 2019. The semi-structured interview protocol is displayed in Figure 2 [19].
3. Results

3.1. Overview of Selected Publications for Content Analysis

In step 4 of the literature review, 62 published articles were selected for content analysis, which included papers from the last 17 years, with more focus on the most recent publications, as shown in Figure 3.

![Figure 3. Number of Selected Papers for Content Analysis by Publication Year](image)
From the context perspective, the selected papers covered 63 countries, with more papers published in the European context, as shown in Figure 4. A review of the literature published during the last 17 years showed the absence of research related to measuring UIC performance in Gulf Cooperation Council Countries, including Qatar. This issue provided a rationale for conducting interviews and refining the findings according to country-specific considerations.

3.2. UIC Performance Explained

University–construction industry relations influence all three institutional paradigms involved in the collaboration (i.e., university, construction industry, and cooperation or collaborative forms of organising). A study of the relationships between the competence factors of universities and UIC performance suggested the significant influence of university research capacity on driving performance of UIC in terms of providing full-time faculty members and the size of the technical licensing office [20]. Moreover, the performance in this study was mediated by government funding for research and development activities. Similarly, another study for the role of technology transfer offices (TTOs) in establishing successful university–construction industry partnerships suggested a positive influence of the university’s social capital on setting research and development (R&D) contracts. Universities with larger social networks, both local and international, were more successful in attracting R&D contracts [21].

The technology parks’ influence on university and construction industry performance differs depending on the university’s involvement and its share in the park. For that reason, it was suggested to distinguish four types of parks [22]. The first type, where the university is the major shareholder, is called a pure science park. The second type, where the university is the minority shareholder, is called a mixed park. The third type, where some university research facilities are located in the park, but the university holds no share, is called a technology park. The fourth type, where the university has no formal involvement, is called a pure technology park. The higher involvement of the university (i.e., the case of the pure science parks) was correlated with the best patenting performance among universities. Surprisingly, such involvement was correlated with the lowest product innovation levels measured by sales from new-to-market products. On the other side of the spectrum, it was suggested that the lowest university involvement (i.e., the case of pure technology parks) was correlated with the lowest patenting performance among universities. On the contrary, such involvement was correlated with the best product innovation levels measured by sales from new-to-market products. However, there is no evidence that the chance of cooperation between universities and the construction industry, in the case of research contracts, has been influenced by the degree of university involvement in park shareholding. Therefore, a high level of patent applications and potentially published research, in the case of pure science parks, does not necessarily lead to increased product innovation. On the other hand, a high level of product innovation, in the case of pure technology parks, is not necessarily associated with a high level of patent applications or published papers. These findings illustrate the dilemma of different interests in university–construction industry relationships. As such, changes to the academic reward system may shift the academic focus from publishing and patenting to including commercialisation [22].

Indeed, with respect to the extent of UIC success in terms of the capability to attract funding for research activities, scholars suggested three important elements that can improve such capability. The first is previous experience with the business [23], which improves maturity and trust-building [24]. The second is the ability to produce a critical mass of research in a certain sector [23], which responds to institutional pressure to improve the performance of the university in research activities [25]. The third is proximity to industrial districts [23], which relates to different types of the distance between university and construction industry partners, including cognitive, geographical, organisational, and social distance [26].

The influence of publicly-funded UIC on R&D efforts by the construction industry provided interesting insights into the construction industry [27]. First, university–construction industry partnerships have a positive impact on the R&D expenditure per employee. Second, university–construction industry partnerships have a positive impact on the share of R&D employment. In fact, partnerships between the university and the construction industry enhance resource utilisation for the construction industry and encourage the construction industry to invest more in R&D activities. University–construction industry relationships are two-way relationships. Thus, evaluations of performance are realised on both sides of the relationship. This is a major break away from the traditional role of university knowledge transfer, namely a one-way relationship. The rise and adoption of the open innovation concept have contributed to a changing paradigm with respect to university–construction industry relations. In the context of UK universities, a recent study revealed that universities had become a central actor in open innovation ecosystems through acting as a reliable intermediary or an open innovation hub [28].

Similarly, studying how to improve the performance of academic innovation in UIC suggested a positive relationship between both formal management mechanisms and regulation implementation and academic innovation [29]. Both relationships were found to be moderated by the university’s innovation climate [29]. In this context, the formal management mechanism for university–construction industry relationships are related to formal arrangements.
to control and coordinate university–construction industry relationships in terms of the university. Regulation implementation is related to the implementation of specific regulations to foster R&D, as well as university–construction industry relations. Finally, the innovation atmosphere reflects the university’s support for entrepreneurial activities by faculty members, students, and administration staff.

Similarly, a recent study, conducted in Europe, found a positive relationship between four management mechanisms and seven key activities of university-business collaboration [4]. The mechanisms were top management support, communication, incentives, and support structures. The seven key collaboration activities were joint curriculum design and delivery, lifelong learning, student mobility, professional mobility, joint R&D, entrepreneurship, and commercialisation of joint R&D results.

The role of academic engagement in sustaining university–construction industry relationships was investigated by Perkmann studies [16]. Academic engagement refers to knowledge-related collaboration activities by researchers with the construction industry and non-academic organisations. The determinants that lead to academic commercialisation are distinguished from the determinants of academic engagement. Commercialisation refers to the use of knowledge created by the university through patenting, licensing of inventions, and business entrepreneurship. Their findings suggest a positive relationship between some individual determinants and academic engagement [16]. These determinants include gender (male), seniority, previous government grant experience, previous construction industry contract experience, and scientific productivity.

Moreover, the recent studies revealed that some other organisational and institutional determinants positively moderate the relationship between individual determinants and academic engagements. These include the university’s focus on applied disciplines. Surprisingly, the quality of the university or department concerned has a negative influence on academic engagement. This finding may be justified by the fact that lower-quality departments often have fewer resources and more reasons to seek engagements and collaborations with the construction industry [16].

In addition, Perkmann the previous studies showed that commercialisation has a positive relationship with individual determinants, which include gender (male), previous commercialisation experience, and scientific productivity [16]. Similarly, some other institutional determinants positively moderated the relationship between individual determinants and academic commercialisation. These include the quality of the university or department concerned, organisational support, organisational commercialisation experience, peer effects, the university’s focus on applied disciplines, and country-specific regulatory policies. Likewise, it was argued that academic commercialisation often leads to increased secrecy and scientific productivity among academics. Finally, academic engagement often leads to improved collaborative behaviour [30]. Moreover, non-academic work experience positively influences external interaction activities among academics [31].

To acquire and share knowledge is vital for both universities and the construction industry, especially in regard to patenting and licensing new technologies [32, 33]. Indeed, university–construction industry relationships influence the performance and outcomes of both institutional paradigms. From the university side, the intended outcomes include attracting third-party funds for employees, research, and operational expenses; research papers published; conferences; presentations; and increased reputation among the scientific community. From the construction industry side, the intended outcomes include new inventions for products, services and processes; new licences; and new patents.

However, there are control factors to examine the impact of knowledge sharing on achieving the objectives of UIC [33]. These control factors include obligations to get external research funding [34], professors’ attitudes towards UIC [35], the degree of applied research [36], the number of employees under professors’ supervision, professors’ years of experience, size of the partner organisation, and the type of partner organisation (i.e., whether it is a private company, a public organisation, or a not-for-profit organisation).

### 3.3. Summary of Key UIC Performance Indicators

Definitions of all performance indicators identified in the content analysis were reviewed, refined, and grouped in order to provide a list of distinct indicators, avoiding duplications. Moreover, measuring UIC performance can take place from the university’s or the construction industry’s perspective. Therefore, the performance indicators from each perspective were grouped into two separate lists.

From university perspective, the literature review revealed performance indicators of UIC. Indeed, the most frequent UIC performance indicators were (1) number of publications [16, 29, 31, 34, 37-51], (2) number of registered patents [3, 7, 16, 20, 29, 37, 40, 48, 50-58], (3) number of occurrences for each UIC activity (consulting and services, research, and training and education) [6, 21, 33, 37, 47, 50, 59-63], (4) number of generated start-ups and spin-offs [16, 37, 40, 48, 50, 53-55, 64-66], (5) amount of external funds to research projects [23, 33, 40, 41, 48, 55, 63, 67], (6) income from intellectual property (IP) sales [20, 37, 48, 50, 51, 54, 65, 68], (7) citation index [16, 31, 39, 44, 50, 69], (8) number of patent applications [37, 50, 52, 54, 70, 71, 78], (9) number of IP licenses [3, 37, 47, 48, 54, 65], and (10) number of innovations (process/marketing/product/organisational) [47, 52, 68].
From the construction industry’s perspective, the literature review revealed performance indicators of UIC. The most frequent UIC performance indicators were (1) number of innovations (process/marketing/product/organisational) [2, 33, 37, 52, 59, 62, 68, 72-79], (2) number of registered patents [33, 54, 56-58, 73, 75, 80, 81], (3) income from innovations (process/marketing/product/organisational) [22, 73-75, 78, 82], (4) number of patent applications [2, 22, 54, 75, 78], and (5) number of publications [37, 73, 83, 84].

The above results show the different focus of the university and construction industry when measuring UIC performance. These findings are in line with the initial discussion stating that universities and industries often operate in different institutional paradigms [28, 85]. However, these results show that both universities and the construction industry share some interests when it comes to four key performance indicators. These performance indicators are (1) the number of registered patents, (2) the number of patent applications, (3) the number of innovations (process/marketing/product/organisational), and (4) the number of publications. These findings suggest that the core of mutual benefit foreseen from university–construction industry relationships are driven by the innovation agenda, represented by patenting activities and being a pioneer in creating knowledge [86].

3.4. UIC Performance Indicators and Context-Specific Considerations

The two semi-structured interviews conducted with scholars revealed interesting findings with respect to measuring UIC performance from the perspective of Qatar University. In order of significance, both interviews highlighted key indicators for measuring UIC performance, including (1) the number of publications, (2) the number of citations (local and international), (3) the number of registered patents, (4) the number of patent applications, (5) the number of training programs provided (to faculty and students), and (6) the number of innovations (process/marketing/product/organisational).

These findings, at least from the interviewees’ perspective, provide some key context-specific considerations. On the one hand, the number of publications and patent registrations and applications are the top UIC performance indicators, which are in line with previous research findings [32]. On the other hand, the number of citations is also related to previous indicators obtained from the literature [32, 71]. Moreover, the indicator of the number of innovations was also indicated by previous studies [52, 68]. However, the high importance of this indicator, from the interviewees’ perspective, may be related to the direction of the Qatar National Research Strategy (QNRS) [10] that aims to put Qatar on the world map as a research and innovation hub [87].

Another interesting finding is the indicator of the number of training programmes provided. From both the university and construction industry perspectives, this indicator is related to the number of occurrences for each UIC activity [21, 33]. The UIC activity, in this case, is collaborative training and education [5].

Historically, university–construction industry relationships have been viewed as a means of transferring knowledge from the knowledge economy (i.e., universities) to the commercial economy (i.e., construction industry and business) [12]. As a result, the latter indicator (number of training programmes provided) may suggest that the interviewees also look to construction industry relationships as a means to transfer knowledge and technology back from the construction industry to university members [8].

In fact, none of the UIC performance indicators obtained from the interviews related to commercialisation and revenue generation. The latter insight is not in line with the top UIC performance indicators obtained from the literature, such as the number of generated start-ups and spinoffs [40], the number of and income from IP sales [65], and the amount of external funds to research projects [21]. Therefore, a deeper look into the motives and drivers of the QNRS [11] is worth further investigation.

4. Conclusions

First, this study revealed that universities are increasingly focused on measuring the performance of their collaborations with the construction industry. The results revealed a number of UIC performance indicators used in previous research, including (1) the number of publications, (2) the number of registered patents, (3) the number of occurrences for each UIC activity (consulting and services, research, and training and education), (4) the number of generated start-ups and spinoffs, (5) amount of external funds to research projects, (6) income from IP sales, (7) citation index, (8) the number of patent applications, (9) the number of IP licenses, and (10) the number of innovations (process/marketing/product/organisational).

Second, this study revealed that the construction industry is increasingly focused on measuring the performance of its collaboration with universities. The results presented a number of UIC performance indicators used in previous research, including (1) the number of innovations (process/marketing/product/organisational), (2) the number of registered patents, (3) income from innovations (process/marketing/product/organisational), (4) the number of patent applications, and (5) the number of publications.
Third, universities and the construction industry share common interests when it comes to driving innovations as a key motive for UIC. This proposition is supported by the common indicators used for measuring UIC performance, namely (1) the number of registered patents, (2) the number of innovations (process/marketing/product/organisational), and (3) the number of patent applications.

Fourth, this study contributes to a general understanding of how universities and the construction industry measure UIC performance. It also highlights Qatar’s country-specific considerations for measuring UIC performance. In this context, the key UIC performance indicators mentioned by the interviewees include (1) the number of publications, (2) the number of citations (local and international), (3) the number of registered patents, (4) the number of patent applications, (5) the number of training programmes provided (to faculty and students), and (6) the number of innovations (process/marketing/product/organisational).

Finally, considering the research limitations, there is also a need for more research examining a wider range of performance indicators and the impact of research strategies and policies on UIC performance. In Qatar, in particular, there is a lack of empirical studies concerning the effects of the QNRS [11] on what indicators are chosen to monitor UIC performance.

In conclusion, despite numerous studies conducted on measuring the performance of UIC, there is still a great deal of research yet to be conducted in order to model, theorise, and empirically test the indicators used to measure UIC performance in general and in the context of the state of Qatar, in particular.

4.1. Limitations and Future Research

While noting the important contributions made by this paper, we recognise a number of limitations. This study is limited to a 17-year period of time and the specific publication databases used for searching for papers. The keywords used in this research provided another limitation regarding the number of papers revealed in the selected publication databases. Thus, it is recommended for future research to consider a wider literature search in terms of time span, publication databases, and keywords.

In addition, only two interviews were conducted with active scholars in UIC. The objective of conducting interviews was to map and refine the findings of the literature review with country-specific considerations. Furthermore, both interviews were conducted with scholars from Qatar University. None of the interviewees came from the construction industry or business. Therefore, mapping and refining the indicators and country-specific considerations from the construction industry perspective was not possible. Consequently, it is recommended to future researchers to consider a wider representation of scholars and business leaders engaged in university–construction industry relations in Qatar, or another context of interest, to get insights that further enable the refinement and mapping of the findings from the literature.

Indeed, this research was mainly a literature review, and country-specific considerations were obtained from a limited number of interviews. Therefore, the results are yet to be confirmed empirically in the context of Qatar. Comparative analysis of UIC performance indicators used across countries is another attractive field for future research.

5. Declarations

5.1. Author Contributions

Conceptualization, Z.A., A.A.S. and M.E.Y.; writing—original draft preparation, Z.A., A.A.S. and M.E.Y.; writing—review and editing, Z.A., A.A.S. and M.E.Y. All authors have read and agreed to the published version of the manuscript.

5.2. Conflicts of Interest

The authors declare no conflict of interest.

6. References


