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Corrosion Inhibition of Sodium Silicate with Nanosilica as Coating in Pre-Corroded Steel

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Abstract
This study was conducted to investigate the potential of using sodium silicate with nanosilica as a treatment to inhibit the progress of corrosion in steel specimens that are already corroded. Steel specimens measuring 16 mm in diameter and 4 mm in thickness were prepared and subjected to pre-corrosion by immersion to 3.5% NaCl solution. Two sets of specimens were then dip-coated with sodium silicate containing nanosilica. One set was coated with 1% nanosilica, and the other was coated with 2.5% nanosilica. The coated specimens were then subjected to Complex Impedance Spectroscopy (CIS) at 20 Hz to 20 MHz frequency range. Compared with the sodium silicate coating with 1% nanosilica, the sodium silicate coating with 2.5% nanosilica had a larger semi-circle curve in the Nyquist plot. Similarly, the sodium silicate coating with 2.5% nanosilica also showed larger magnitudes of impedance at the low-frequency region and larger phase angles at the high-frequency regions in the Bode plot. These results imply that the sodium silicate coating with 2.5% nanosilica coating demonstrated better capacitive behavior. In addition, equivalent circuit modelling results also showed that the sodium silicate coating with 2.5% nanosilica had higher coating resistance and lower coating capacitance as compared to the sodium silicate coating with 1% nanosilica.

Keywords: Corrosion; Coating; Nanosilica; Impedance Spectroscopy.

1. Introduction
Corrosion is an electrochemical action that is primarily due to chloride ingress. Corrosion of structural elements affects different structures in various ways. In reinforced concrete structures, corrosion products or rust accumulate in the area of reinforced concrete, leading to internal stress which leads to concrete cracking or spalling. Corrosion in steel structures causes reduction in the area of the structural members, leading to lower resistance to stresses [1]. On a global scale, damages that are caused by corrosion are approximately $276 billion per year [2]. An example of which was the repair and

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demolition of Silver bridge in Ohio, USA, in 1967 [3]. With a 17.8% increase in the price index of steel from 2012 to 2018 [4], protection of existing structures against corrosion damages has been the more viable option over reconstruction.

The most commonly used method in treating corroded steel is through the use of blasting and application of epoxy coating. Blasting uses abrasive materials to remove the existing rust produced by corrosion, therefore effectively cleaning the steel substrate [5]. It is also effective in removing contaminants on the steel surface, such as oil and grease. Furthermore, blasting creates a profile on the steel surface that is suitable for applying the protective coating. After blasting, application of the protective coating follows. Coatings such as epoxy coating act like a barrier preventing the intrusion of harmful elements like chlorides that causes corrosion of the steel. It also provides a resistant media between the anodic and cathodic portions found on the steel surface [6-7]. This protective property of coating is important because defects on a coated steel surface are difficult to prevent. Once a defect is formed on the coated surface, corrosion process can occur between the defect area and the sound portions. The progress of this type of corrosion process will be dependent on the resistance of the coating used [8-9].

Inorganic material-based nanoparticles have been extensively studied as effective and more environmentally friendly coating components for the past years. Past research works explored the use of various nanoparticles, such as Fe₃O₄, ZrO₂, ZnO, and SiO₂ to enhance the mechanical properties of anti-corrosion composite coatings [10-13]. In a study by Khan et al. [14] for epoxy-based coatings, it was found out that among the nanoparticles, nanosilica increased the hardness and elastic modulus the most by 71% and 26%, respectively.

Nanosilica has been used to enhance the properties of composites due to its structure and morphology [15]. Rice Hull Ash (RHA), an agro-industrial waste from rice production, contains over 90% nanosilica thereby making extraction of nanosilica from RHA highly cost-efficient [16-17]. Studies also have shown that nanosilica can prevent corrosion through the formation of a silicate network [18]. Its nanoscale size allows it to penetrate small voids formed by corrosion products. Moreover, high surface area causes nanosilica to be easily dissolved in a media that contributes to silicate network formation's uniformity [19]. The silicate network forms thin layers that can act as a physical barrier that prevents chlorides and contaminants that may initiate corrosion. Alkali silicate anti-corrosion coating is also known as silicate conversion coating. It is one example of an alkali-treated anti-corrosion coating. The structure of the coating contains polymeric particles such as silanol groups (Si-OH) and siloxane bridge (Si–O–Si) [20]. During the curing process, the polymerization continued and more siloxane groups were converted to siloxane bridges [21].

Dela Cruz et al. [22] studied the use of nanosilica and polyaniline (PANI) composites in epoxy coated steel specimens. The study showed that all percent combinations of nanosilica and PANI composites in the epoxy coating have better corrosion protection performance compared to that of pure epoxy coated steel by yielding higher coating resistance and lower coating capacitance. Among the different percent combinations of nanosilica-PANI, the 40-60% combination of nanosilica and PANI exhibited the highest coating resistance and lowest coating capacitance. In another study, the group of Ruzgal [23] investigated the effect of nanosilica when used as a coating enhancer for cold-rolled steel. Their study demonstrated that nanosilica-enhanced coating reduced soil-to-metal friction and soil adhesion for cold-rolled steel by 24% and 36%, respectively.

To evaluate the performance of coatings and corrosion activity in steel, researchers use a non-destructive technique called Complex Impedance Spectroscopy (CIS). Through CIS technique, the impedance at different frequencies can be measured. The impedance data, particularly those found at the low-frequency region, can characterize the coating resistance and capacitance. At the same time, the impedance data can be used to determine the rate of corrosion of the substrate steel underneath the coating and the failure mechanism of coating systems. The CIS technique was used in this study.

Research on the use of sodium silicate with nanosilica, synthesized from RHA, as a treatment for corroded steel is scarce. This study aims to contribute knowledge and understanding on the potential capability of sodium silicate, with nanosilica from RHA, in treating corroded steel by inhibiting further progress of corrosion. The specific objectives of the study are: (1) to characterize the effectiveness of sodium silicate coating with nanosilica applied to pre-corroded steel by examining the generated Nyquist and Bode plots and (2) to determine the resistance and capacitance of sodium silicate coating with nanosilica using Equivalent Circuit Modelling.

This paper is structured as follows: materials and methods are discussed in section 2. It is in this section where the material properties, preparation of specimens and test methods are presented in detail. In section 3, the results of Nyquist and Bode plots are presented and discussed together with the results of the equivalent circuit modelling. Section 4 presented the conclusions of the study.

2. Materials and Methods

In this study, the corrosion inhibition of sodium silicate with nanosilica synthesized from RHA was evaluated by using CIS in coated steel specimens. The steps for evaluation are shown in Figure 1.
2.1. Materials Used

2.1.1. Steel

Round steel bars that are locally and commercially available were used in the study. Initially, round bars of 16 mm diameter were cut into 4 mm thick coupons and were subjected to surface preparation in accordance with ASTM G1 standard method [24]. Briefly, existing rusts, dirt, and oil were removed by mechanical abrasion with using no. 100 sand paper and subsequently washing with distilled water and acetone. Representative sample specimens used in this study are shown in Figure 2.

![Sample steel specimen used](image)

Morphological and topographic nanoscale analysis were done using the XE-70 atomic force microscope in non-contact mode on the steel specimen's surface. Non-contact cantilever with resonant frequency of 330 kHz, force constant of 42 N/m, length of 125 μm, mean width of 30 μm and thickness of 4 μm were used in the analysis. Various scanning areas of 45×45 μm in size at different portions of the steel specimen surface were obtained at a scan rate of 0.30 Hz.

The analysis of the steel specimen (Figure 3) showed uniform striations and a smooth surface. It was also observed that there are grooves present on its surface, possibly caused by surface preparation through mechanical abrasion using SiC abrasive paper. Presence of bulbous material was immediately observed possibly indicating that a rapid corrosion formation on the carbon steel surface was taking place. This phenomenon is frequently observed on an air-exposed steel surface.
X-ray diffraction analysis was done to further characterize the steel specimen. Shimadzu XRD-6100 equipped with an x-ray source with a copper target (wavelength 1.540598 Å) and has an accelerating voltage of 30 kV with a scan rate of 2° per minute and a scan range of 2° to 90° was used. Post-run analyses were done using X’Pert HighScore Plus software. Figure 4 shows the X-ray diffractogram of a clean steel sample.

### 2.1.2. Nanosilica

The nanosilica, shown in Figure 5, was synthesized from RHA using the technology developed by the group of Dr. Engelbert Peralta from the College of Engineering and Agro-Industrial Technology of the University of the Philippines Los Baños. The physical properties of nanosilica are summarized in Table 1.
Table 1. Physical properties of the nanosilica used in the study

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Average Particle Size</td>
<td>24.39 ± 0.38 nm</td>
</tr>
<tr>
<td>Surface Area</td>
<td>~260 – 300 m²/g</td>
</tr>
<tr>
<td>Pore Radius</td>
<td>~16 – 19 Angstrom</td>
</tr>
<tr>
<td>Pore Volume</td>
<td>~0.246 cc/g</td>
</tr>
</tbody>
</table>

The crystallography of the nanosilica was determined through X-ray Powder Diffraction (XRD). In contrast to the crystallographic analysis of crystalline silica, which peaked at 22°, the crystallographic analysis of the powdered nanosilica used in this study (Figure 6) showed no peak at 22°. This means that the powdered nanosilica used in this study is highly amorphous. In addition, the presence of other contaminants was not observed, which means that the powdered nanosilica used in this study was also highly pure.

Figure 6. Crystallographic analysis of the powdered nanosilica used in the study

2.2. Pre-corrosion of Test Specimens

The cleaned steel samples were pre-corroded by exposing them to 3.5 (weight to volume) % NaCl solution. The samples were submerged in the solution for three days and then air-dried for one day. The pre-corroded steel sample specimen is shown in Figure 7.

Figure 7. Pre-corroded steel specimen

2.3. Preparation Of Sodium Silicate Solution

The nanosilica was oven-dried for 24 hours at 150°C to remove existing moisture. It was then cooled to room temperature. Desiccators were used to ensure a dry environment for the nanosilica. While cooling the nanosilica, a 3M
NaOH solution was formulated. Two 100mL NaOH solutions were used to dissolve 1.0 and 2.5 grams of nanosilica. The solutions were heated for 20 minutes at 80°C. The solutions were then cooled to room temperature.

2.4. Application of Coating in Test Specimen

The specimens were coated with sodium silicate containing nanosilica using dip-coating method with a submersion time of 30 minutes. The coated specimens were then oven-dried at 100°C for 20 minutes.

2.5. Impedance Measurement using CIS

Using an Impedance Analyzer shown in Figure 8, the test specimen’s real and imaginary impedance components were measured and recorded at varying frequencies (20 to 20 MHz). The impedance spectra evolution through time was derived from the assessment of the coating’s impedance using CIS. The total impedance was measured and noted which follows Ohm’s law defined by Equation 1.

\[
Z(w) = \frac{V(w)}{I(w)} = Z_{\text{real}}(w) + Z_{\text{img}}(w) j
\]

CIS was used to generate the Nyquist and Bode plot of the respective coatings. The Nyquist plot was formed by plotting the imaginary impedance component in the y-direction and the real impedance component on the x-direction. On the other hand, the Bode plot was obtained by plotting in a logarithmic scale the total impedance measured at varying frequencies from 20 Hz to 20 MHz. The total impedance was computed using Equation 2.

\[
|Z(w)| = \sqrt{Z_{\text{real}}(w)^2 + Z_{\text{img}}(w)^2}
\]

Figure 8. Impedance Analyzer used in the study

ECM was used to quantify the coating resistance and coating capacitance using EC Lab as the fitting software and Igor as the plotting software. From the total impedance components measured using CIS, the Nyquist plot was generated. Using the Z-fit feature of EC Lab 10.40, the ECM’s graph was fitted to the actual Nyquist plot. The equivalent circuit used to determine the values of each electrical component, particularly the coating resistance and capacitance, is shown in Figure 9.

Figure 9. Equivalent circuit used in the study
3. Results and Discussion

3.1. Nyquist Plot

CIS was employed to evaluate the corrosion inhibition of the sodium silicate coating with 1% and 2.5% concentration of nanosilica. The real and imaginary values of the impedances were plotted to obtain the Nyquist plots. Using the Nyquist plot, the coating performance can be interpreted through the position and size of the diameter of the semi-circle formed. The semi-circle plot showing an almost vertical behavior near the vertical axis, and that with a larger diameter signifies better coating performance.

Figure 10 shows the Nyquist plots of sodium silicate coated steel obtained with 1 and 2.5% nanosilica. The Nyquist plot for the sodium silicate coated steel containing 2.5% nanosilica show more vertical behavior and larger diameter thus demonstrating better quality compared to the coating with 1.0% nanosilica. It illustrates the capacitive behavior of the coating commonly observed in coated steel substrate that is not experiencing corrosion. On the other hand, the Nyquist plot for the sodium silicate coated steel containing 1.0% nanosilica shows a more depressed loop. A Warburg impedance started to appear as a diagonal line at the end of the loop which demonstrates diffusion induced behavior. With this observation, it is important to include a Warburg element in the equivalent circuit model during curve fitting.

![Figure 10. Nyquist plot of coated steel specimens](image)

3.2. Bode Plot

The Bode plot was generated by plotting the logarithm of the impedance modulus and the phase angle displacement as functions of the logarithm of frequency. The impedance modulus and phase angle displacement at different frequencies are shown in Figures 11 and 12, respectively. It can be observed that the coating with 2.5% nanosilica exhibited higher impedance values at the low-frequency region and higher phase angle at the high-frequency region as compared to the coating with 1.0% nanosilica. Therefore, the coating with 2.5% nanosilica exhibited a more capacitive behavior compared to the coating with 1.0% nanosilica.
3.3. Equivalent Circuit Model

Equivalent circuit model-fitting has been used as a powerful technique to get insights into the properties of a variety of materials [25]. The CIS data of the coating systems were fitted graph of the equivalent circuit in order to calculate the values of coating capacitance (Cc) and coating resistance (Rc). The curve fitting is shown in Figure 13 while the computed values of the coating capacitance and coating resistance are shown in Table 2. The sodium silicate coating containing 2.5% nanosilica showed lower capacitance than the 1% nanosilica coating. This indicates that the coating with 2.5% nanosilica has a lower electrolyte intake and is less porous than the 1% nanosilica coating. In addition, a higher coating resistance was observed for the sodium silicate coating containing 2.5% nanosilica. The increase in resistance was found to be one order in magnitude. This indicates that the coating with 2.5% nanosilica has a better corrosion inhibition for the steel substrate.
In this study, the steel substrate was already subjected to corrosion prior to application of sodium silicate with nanosilica as coating. Therefore, it is important to note that the surface of the substrate is already non-uniform and contains crevices and voids as a consequence of the induced corrosion. Without a barrier protection, these crevices and voids will serve as venues where local corrosion can occur and progress.

The sodium silicate coating used in this study formed a thin layer of hardened paste on the surface of the corroded steel substrate. This finding is in congruence with the work of Ruzgal et al. [23] where the hardened paste composed of silica hydrates had a tendency to be accumulated and deposited in the crevices of the steel substrate. The tendency of the sodium silicate hydrate to fill the crevices present in the steel substrate is due to its high surface area-volume ratio. The accumulation of silica in the crevices provides an indicative explanation why the coating with 2.5% nanosilica exhibited better corrosion inhibition performance compared with the coating with only 1% nanosilica.

### 4. Conclusion

The study investigated the performance of sodium silicate on preventing the progress of corrosion in pre-corroded steel specimens. CIS technique was used to evaluate and compare the coating performance of sodium silicate with 1 and 2.5% nanosilica derived from RHA. Both the Nyquist plot and Bode plots provided evidence that the sodium silicate with 2.5% nanosilica derived from RHA has prevented the progress of corrosion in coated steel plates. The Nyquist plot for the sodium silicate with 2.5% nanosilica exhibited a semi-circle with a larger diameter than the semi-circle plotted using the coating with only 1% nanosilica. These results exemplify a capacitive coating behavior. In terms of the Bode plot, the sodium silicate with 2.5% nanosilica exhibited a higher magnitude of impedance at the low-frequency region and larger phase angles at the high-frequency regions, which are also indicators of a capacitive behavior. Equivalent circuit modelling enabled the comparison of the values of coating resistance and coating capacitance of sodium silicate with nanosilica derived from RHA. Sodium silicate with 2.5% nanosilica had a higher coating resistance by one order in magnitude. In terms of coating capacitance, the coating with 2.5% nanosilica also showed lower capacitance indicating that it has lower electrolyte intake and is less porous compared to the coating with only 1% nanosilica. The study therefore has provided evidence that sodium silicate with nanosilica derived from RHA can inhibit the further progress of corrosion in steel substrate. In addition, sodium silicate with 2.5% nanosilica derived from RHA performs better than a sodium silicate with only 1% nanosilica derived from RHA in inhibiting the progress of corrosion in steel substrate.

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**Table 2. Coating capacitance and resistance values**

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<th>Cc (Farads)</th>
<th>Rc (Ohms)</th>
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<tr>
<td>1.0% NanoSiO₂</td>
<td>2.37 E-09</td>
<td>4.38 E+04</td>
</tr>
<tr>
<td>2.5% NanoSiO₂</td>
<td>2.93 E-10</td>
<td>1.16 E+05</td>
</tr>
</tbody>
</table>

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5. Declarations

5.1. Author Contributions


5.2. Data Availability Statement

The data presented in this study are available in article.

5.3. Funding

This work is primarily funded by the Philippine Council for Industry, Energy and Emerging Technology Research and Development, Department of Science and Technology with Project No. 03713, 2020.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

6. References


Uncertainty Analysis of Regional Rainfall Frequency Estimates in Northeast India

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Abstract

Estimation of rainfall quantile is an important step in regional frequency analysis for planning and design of any water resources project. Related evaluations of accuracy and uncertainty help to further assist in enhancing the reliability of design estimates. In this study, therefore, we investigate the accuracy and uncertainty of regional frequency analysis of extreme rainfall computed from genetic algorithm-based clustering. Uncertainty assessment is explored with prediction of quantiles with a new spatial Information Transfer Index (ITI) and Monte Carlo simulation framework. And, accuracy assessment is done with the comparison of regional growth curves to at-site analysis for each homogenous region. Further, uncertainty assessment with the ITI method is compared with Maximum Likelihood Estimation (MLE) optimized by a Genetic Algorithm (GA) to check the suitability of the method. Results obtained suggest the ITI-based uncertainty assessment for regional estimates outperformed those of at-site estimates. The MLE-GA method based on at-site estimates was found to be better than at-site estimates based on L-moments, suggesting the former as a better alternative to compare with regional frequency estimates. Moreover, minimal bias and least deviation of the regional growth curve were obtained in the rainfall regions. The confidence intervals of regional estimates were seen to be well within the bounds of normality assumptions.

Keywords: L-Moments; Monte Carlo; Information Transfer Index; MLE; GA; Rainfall.

1. Introduction

The Brahmaputra and Barak basins in northeastern India are among the country's most disaster-prone locations, with severe rainstorms and cloud bursts occurring annually during the monsoon season. The frequency of extremely heavy downpours in the basins has been shown to fluctuate widely over the region. As a result, human life and property are seriously damaged, which has an impact on the region's total socioeconomic activity. With a limited network of rain gauge stations, flood related information and mitigation measures have always been insufficient in the region. Methods such as Regional Frequency Analysis (RFA) have been frequently employed in such situations that transfer data from gauged locations to places with little or no data [1, 2]. Several studies can be found with the application of the RFA technique in the region [3-7]. Regional frequency analysis of extreme rainfall for any region aims to provide a detailed description of the distribution of rainfall events and predict probable estimates for a given return time. However, the

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application of the RFA technique is always associated with some amount of uncertainty. Quality of data, number of available records, and processes involved in any regional frequency analysis (RFA) are important sources of uncertainty, and so is crucial to analyse the uncertainty inherent in their applications. The fitting of the probability distribution and parameter estimation is an important factor affecting RFA analysis [8, 9]. With uncertainty and suitability involved in the selection of an appropriate probability distribution for any regional frequency analysis, it is crucial to have a performance comparison of fitted distributions against at-site analysis for the region. Numerous works on uncertainty analysis in regional frequency analysis of rainfall [10-13] have been published. Notable recent studies on the comparison of uncertainty analysis of rainfall from regional and at-site analysis can be found in the studies of [14-16], but very scarce studies are available for north east India. The available studies are limited to only selection of probability distribution and estimation of quantiles [3, 5]. Resulting quantiles from RFA with small sample sizes are speculative, and constructing confidence intervals has been the simplest and most widely used approach to evaluate the uncertainty. There is rarely any comparison study of uncertainties associated with at-site and regional analysis in the study region, which is a necessary area to be explored. The study will give an assessment of the extreme rainfall behaviour with respect to fitted distributions both at regional and at-site levels.

Besides evaluating confidence intervals based on Monte Carlo simulation, several other approaches for analysing uncertainty have been developed, including generalised likelihood uncertainty estimation (GLUE), the Bayesian method, and Markov chain Monte Carlo (MCMC), among others [17-21]. The majority of them were used to quantify uncertainty in hydrological models. But the approach for analysing uncertainty for at-site and regional frequency analysis using the Monte Carlo simulation and entropy-based information transfer index (ITI) framework till date has not been explored. In this study, the uncertainty is performed with generation of new samples using Monte Carlo simulation from entropy dependent weights at an unmeasured site for rainfall estimates. Compared to other uncertainty methods, there is rarely any such study of the present approach in frequency analysis. The entropy concept has been used in many hydrological studies [22-25], but its application in uncertainty analysis in RFA has not been done. The proposed method is based on the idea that information at a new station is more accurately generated from nearby stations when stations with a higher amount of shared information are selected rather than stations based on proximity or distance.

The majority of the regional frequency analysis studies conducted in India’s northeast area concentrated on the Brahmaputra basin or important chosen stations from the whole northeast region. However, relatively little research on regional frequency studies of yearly extreme rainfall, including rain gauge stations from the Barak basin, is available. During the monsoons, the Barak basin is severely prone to flooding, producing flood issues comparable to those seen in the Brahmaputra basin. So, with the inclusion of stations from the Barak basin in the study, will provide a more comprehensive and enlarged perspective of the extreme rainfall scenario in the northeast area. Moreover, to the best of our knowledge, research on the uncertainty of regional rainfall quantiles for homogenous rainfall regions in the Brahmaputra and Barak basins is scarce. The study therefore aims to assess the uncertainty of extreme rainfall estimates derived from regional frequency analysis and to perform a comparison with at-site analysis. To assess the suitability of design quantiles, two approaches are investigated: (i) uncertainty estimation using coefficient of variation of rainfall estimates and the development of confidence intervals; and (ii) using the framework of ITI and Monte Carlo simulation and comparing it to at-site frequency. Furthermore, to investigate the applicability of ITI-based weight determination with different parameter estimation methods, MLE estimation optimised by a genetic algorithm (GA) is investigated.

2. Study Area and Rainfall Data

The study region in northeast India concentrates on the southern part of the Brahmaputra basin and the Barak basin. The Barak basin, just beside the Brahmaputra basin, has active floodplains with vast marshy regions that are annually inundated by severe floods. The altitudinal pattern in the north east changes drastically from location to location, resulting in erratic rainfall occurrences. During the monsoon season, heavy rains and cloud bursts are common, wreaking havoc on the region and causing widespread landslides and erosion. Amount of precipitation is particularly unpredictable in the region, making future rainfall scenarios highly vulnerable. For the study, annual maximum daily rainfall data for 33 stations is collected from the Regional Meteorological Centre, Guwahati, for a period of 20 years, and their locations in the research region are given in Figure 1. The list of stations included in the study are Silchar (1), Dholai (2), Goalpara (3), Guwahati (4), North Lakimpur (5), Choudhoughat (6), Batadighat (7), Kampur (8), Sibsagar (9), Beki Rd. Bridge (10), Dibrugarh (11), Jorhat (12), Neamatighat (13), Kherunighat (14), Bokajan (15), Gossaigaon (16), Kokrajhar (17), Tezpur (18), Mellabazar (19) Aie NH.Xing(20), Dharamtuli(21), Golaghath(22), Dhillabazar (23), Margherita (24), Gharmura (25), Shillong (26), Cherrapunjee (27), Mawsynram (28), Kohima (29), Imphal (30), Aizwal (31), Agartala (32), and Kailashahar (33).
3. Research Methodology

3.1. Formation of Homogenous Regions and Heterogeneity Measurements

The study used genetic algorithm-based clustering to designate homogeneous rainfall areas, with the Davies-Bouldin index as the fitness function. Based on the multi-criteria decision technique and heterogeneity measures proposed in [2], three optimal station groups were determined and found to be homogeneous. Clustering was done using seven station characteristics: latitude, longitude, altitude, annual daily maximum average, greatest annual daily maximum, lowest annual daily maximum, and annual maximum series coefficient of variation. Prior to clustering, the variables were standardised using the max-min transformation. Verification of the homogeneity of identified homogenous regions in regional frequency analysis is very important and is done in the present study using the heterogeneity measure [2]. According to Tasker et al. (1998) [2], a region is declared acceptably homogenous when $H<1$, a possibly heterogeneous when $1 \leq H < 2$ and definitely heterogeneous if $H \geq 2$. More information about the procedure can be obtained from [2]; and, the study framework for comparing regional and at-site precipitation frequency analyses is summarised in Figure 2, which addresses the steps followed in determining uncertainty.

3.2. Choosing Best Fit Distribution and Accuracy of Quantiles

The best-fit probability distribution for all three regions was determined by testing five three-parameter candidate distributions: generalised normal (GNO), generalised Pareto (GPA), generalised extreme value (GEV), generalised logistic (GLO), and Pearson type 3 (PE3). The goodness-of-fit metric as suggested in [2] for homogenous regions is considered to find the best distribution and is calculated as

$$\left| Z_{\text{dist}} \right| = \frac{(\tau_4^{\text{dist}} \beta - \tau_4^R + \beta_4) / \sigma_4}{\tau_4^R}$$

where “dist” is the candidate distribution; $\tau_4$ the regional average L-kurtosis value calculated in simulation; $\beta_4$ and $\sigma_4$ the bias and standard deviation respectively of regional average L-kurtosis ($\tau_4^R$) of Monte Carlo simulation samples performed by Kappa distribution. For all values of $\left| Z_{\text{dist}} \right| \leq 1.64$, the corresponding candidate distributions are considered fit and acceptable at 90% confidence level. And to decide on the best-fit distribution, the candidate distribution with lowest $\left| Z_{\text{dist}} \right|$ is selected as the best distribution for the region. The annual maximum rainfall quantiles for different probabilities of non-exceedance $F$ are then calculated with selected fitted distributions using the method of index flood approach.
To evaluate robustness of the regional quantiles for each region, the procedure mentioned by Tasker et al. (1998) [2] involving generation of regional average $L$-moments from Monte Carlo simulations is used. The simulation involves generation of quantile estimates for various return periods, and at a given $m^{th}$ repetition, the estimated quantiles for a given non-exceedance probability $F$, $\hat{Q}_i[F]$ is estimated and compared with true values of $Q_i(F)$. The relative error of this estimate at a given site $i$ and for a non-exceedance probability $F$ is expressed as:

$$\frac{\hat{Q}_i[F] - Q_i(F)}{Q_i(F)} \times 100$$

This quantity is squared and averaged for $M$ repetitions to obtain the relative bias and mean relative quadratic error as:

$$\text{Relative Bias} = \frac{\sum_{i=1}^{M} (\hat{Q}_i[F] - Q_i(F))^2}{M}$$

$$\text{Mean Relative Quadratic Error} = \frac{\sum_{i=1}^{M} (\hat{Q}_i[F] - Q_i(F))^2}{M}$$
\[
B_i(F) = M^{-1} \sum_{m=1}^{M} \frac{\hat{q}^{[m]}(F) - \hat{q}(F)}{\hat{q}(F)} (3)
\]

\[
R_i(F) = \left[ M^{-1} \sum_{m=1}^{M} \frac{\hat{q}^{[m]}(F) - \hat{q}(F)}{\hat{q}(F)} \right]^{1/2} (4)
\]

And the summary performance of all stations in a region, is expressed by regional relative bias and relative root mean square error as;

\[
B^R(F) = N^{-1} \sum_{i=1}^{N} B_i(F) (5)
\]

\[
A^R(F) = N^{-1} \sum_{i=1}^{N} |B_i(F)| (6)
\]

\[
R^R(F) = N^{-1} \sum_{i=1}^{N} R_i(F) (7)
\]

### 3.3. Uncertainty Analysis of Fitted Distribution Parameters and Quantiles

For higher return periods, the quantiles computed have a higher degree of uncertainty. Uncertainty in parameter estimation and its stability are two important types of uncertainty associated with any given quantile prediction. These uncertainties will be examined in the present study for both regional and at-site analysis with the help of a Monte Carlo simulation framework. Furthermore, confidence intervals are developed to compare accuracy of computed quantities both from at-site and regional analysis. In the present study, coefficient of variation is used as a measure of uncertainty to test the parameter stability of identified distributions. A \( C_v \) for a particular quantile is defined as \( C_v = \sigma / \mu \) where, \( \sigma \) and \( \mu \) are the standard deviation and mean of quantiles estimated from various GEV and PE3 distributions. With 1000 Monte Carlo simulations, random distinct sample sets is generated each time, having the same distribution with different parameters.

For the comparison of uncertainty in prediction of regional and at-site analysis in each region, a quantitative indicator is proposed that takes into account the spread of confidence intervals. For a given quantile prediction, an average relative width [26, 27] is used and is given as;

\[
ARW = \frac{1}{n} \sum \text{Limit}_{\text{upper}} - \text{Limit}_{\text{lower}} \frac{1}{q(F)} (8)
\]

where \( \text{Limit}_{\text{upper}} \) and \( \text{Limit}_{\text{lower}} \) are upper and lower limit of corresponding 95% error bounds, \( n \) is the number of stations in a region, \( q(F) \) is the estimated quantile for probability of non-exceedance, \( F \). Smaller value of ARW indicates a smaller uncertainty of the estimated quantile.

The uncertainty of predicted quantiles for different return periods is assessed by constructing confidence intervals. The steps for constructing confidence intervals for each return period of a station are as follows:

- First, the parameters for fitted distribution of each region is determined using method of L-moments based on observed data of each site.
- A set of generated data having the same sample size as of the site \( i \) is obtained.
- Monte Carlo simulation is then carried out and parameters of the fitted distribution for each generated sample is calculated and the precipitation quantiles is estimated.
- The 2.5 and 97.5 percentile values for 95 percent lower and upper bounds are obtained for each return period from each Monte Carlo simulation. Similarly, 95 percent confidence interval boundaries are generated for all other return periods of interest.

The confidence intervals from normality assumption is also constructed for comparison purpose and for a target return period with sample mean \((\mu_T)\) and standard deviation \((\sigma_T)\), the upper and lower bounds at 95% confidence interval is calculated.

### 3.4. Uncertainty Analysis of Parameter Estimation using Entropy based Information Transfer Index

The uncertainty of the parameters identified for probability distributions of clustered regions is assessed using Monte Carlo Simulation and entropy dependent weights. The stability of the identified parameters is studied to assess its capability in identifying extreme rainfall depth at any ungauged site with the help of nearby surrounding sites. The current study differs from previous techniques in [11, 28], in the application of an information transfer index based on entropy to generate new time series for an unmeasured site and the weights proposed herein the study is given as;

\[
w_i = \frac{\text{ITI}_i}{\sum_{i=1}^{k} \text{ITI}_i} \quad (i=1,2,3,...,k) (9)
\]
where, \( w_i \) is the extraction entropy weight from station \( i \), \( k \) is the number of sites. The extraction entropy weight depends on information transfer index (ITI) as expressed by [22] and is given as:

\[
ITI(A, B) = \frac{T(A, B)}{H(A, B)}
\]

\[
T(A, B) = H(A) + H(B) - H(A, B)
\]

where; \( H(A) \) and \( H(B) \) denote the marginal entropies of rain gauge stations \( A \) and \( B \), respectively, whereas \( H(A,B) \) denotes their combined entropy. ITI is a symmetric index that quantifies the exchange of information between two stations. The weight value is between 0 and 1, and a greater value suggests a more effective communication of information. The new time series at the ungauged site \( A \) is generated with:

\[
Y_A = \sum_{i=1}^{k} w_i \cdot Y_i \quad (i=1,2,3,...,k)
\]

To assess the performance of the new extraction entropy weight based on ITI it is compared to two other weighting methods viz. one method based on Euclidean distance [29] and the other method is a combination of ITI and Euclidean distance.

\[
w_i = \frac{dist^{-1}}{\sum_i dist^{-1}} \quad (i=1,2,3,...,k)
\]

\[
w_i = \frac{dist^{-1}ITI}{\sum_i dist^{-1}ITI} \quad (i=1,2,3,...,k)
\]

New time series samples equal to the number of data length at each site for \( M \) extraction sites are generated using Monte Carlo simulation using the parameters of the fitted distribution for the site. The new time series samples at an ungauged Site \( A \) can then be generated using Equation 3. For regional analysis, new time series of all sites in the homogeneous region is simulated and taken together. The application of ITI approach in assessing the parameter stability of distributions is also extended to application of Maximum Likelihood Estimation (MLE) optimized by genetic algorithm. The parameter estimation of distributions of individual sites for use in ITI is done by MLE method and optimized by genetic algorithm. And the comparison is done to weighting method based on Euclidean distance. The comparison will help us to understand the applicability of ITI method in determining spatial weights for prediction at ungauged sites. The uncertainty in estimated design rainfall depth is computed as:

\[
\text{Uncertainty (\%)} = \left( \frac{P_{95} - P_5}{P_{50}} \right) \times 100
\]

where \( P_{95}, P_5 \) and \( P_{50} \) are the expected design rainfall depths at the 95th, 5th, and 50th percentiles, respectively. The estimates by the ITI based L-Moment estimates, comparison with MLE to fitting GEV and PE3 distributions is explored. Maximum Likelihood Estimation (MLE) is a frequently used technique for estimating parameters of probability distributions, in which the parameter estimates generate the highest chance of occurrence for observations. With numerous application in extreme value models [12, 30], it is considered in the present study for comparison with estimates of ITI based L-Moment estimates. GA was used for optimizing the MLE parameters to arrive at the likelihood of the real value. GA are population-based algorithms and have successfully provided near-real value solutions in various complex problems. The log-likelihood function of the three parameters of the GEV and PE3 distribution is given as:

\[
\ln[L(x|\theta_1, \theta_2, \theta_3)] = \sum_{i=1}^{n} f(x_i|\theta_1, \theta_2, \theta_3)
\]

where, \( f(x|\theta_1, \theta_2, \theta_3) \) is the pdf of GEV or PE3 distribution and \( x_i = (x_1, ..., x_n) \) are the observations.

The values of the parameters are then obtained by partially differentiating the log-likelihood function with respect to each parameter and equating it to zero. To further assess the performance of ITI, distance-based and MLE-GA estimates for both regional and at-site.

4. Results and Discussion

The rain gauge stations were tested for trend and randomness of data series using the Mann-Kendall and Ljung Box tests [31]. The study results indicate no trend, and the data were serially independent, making them acceptable for statistical frequency analysis and fitting of probability distributions. The grouping of gauge stations using genetic algorithm based clustering and Euclidean distance measure resulted in three regions. Nine cluster validation measures and MCDM analysis gave three homogenous regions and is given in Table 1. Heterogenity test as proposed by [2] was applied and the final homogenous regions I, II and III composed of 9, 2 and 20 stations respectively after removal of
two discordant stations Golaghat and Goalpara. Applying the goodness of fit criterion, the fitted frequency distributions for the three regions are selected and presented in Table 1.

Table 1. Homogenous regions obtained from genetic algorithm-based clustering

<table>
<thead>
<tr>
<th>Regions</th>
<th>No. of Stations</th>
<th>Stations</th>
<th>Heterogeneity Measure</th>
<th>Distribution function</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>9</td>
<td>North Lakhimpur, Choudhoughat, Bokajan, Sibsagar, Margherita, Dibrugarh, Jorhat, Dhillolabazar, Neamatighat</td>
<td>0.33 0.92 0.37</td>
<td>GEV</td>
</tr>
<tr>
<td>II</td>
<td>2</td>
<td>Cherrapunjee, Mawsynram</td>
<td>-0.54 -0.83 -0.15</td>
<td>GEV</td>
</tr>
<tr>
<td>III</td>
<td>20</td>
<td>Silchar, Dholai, Guwahati, Kampur, Kherungibhat, Imphal, Batadghat, Gossaigaon, Kokrajhar, Tezpur, Mellabazar, Aie N. H. Xing, Dharamtul, Gharma, Beki Road Bridge, Shillong, Kohima, Aizwal, Agartala, Kailashahar</td>
<td>0.06 -1.11 -1.83</td>
<td>PE3</td>
</tr>
</tbody>
</table>

4.1. Estimation of Precision of Regional Quantile Estimate

In this section, the precision of dimensionless regional growth curve q(F) for each homogenous region is calculated and shown in Table 3. With 10,000 simulations, Monte Carlo simulations procedure was carried out with the selected distribution for each region. In the process, the simulated regional quantiles were compared to the real data for all non-exceedance probability to obtain the precision measurements. Regional relative root mean square error R^2(F) computations show that region III has the least deviation in regional growth curve with slightly higher values obtained in region II. This is an important criterion as it signifies the overall deviation of difference between computed quantiles and true quantiles of all stations in a region. As region II comprises of only two stations and the annual maximum rainfall average are comparatively very high, the bias in estimated regional growth curve may be affected by sampling of a smaller number of stations. B^2(F) and A^2(F) also suggests minimal difference between simulated and true quantiles for all return periods in all the three regions. A^2(F) which gives a measure of bias of estimates of quantiles to be consistently high at some stations and low at others [32] is found to be least in region I, and hence the accuracy of quantile estimates in this region will be better among the three regions. Overall, from the analysis, the estimated regional quantile growth curve in each region is found to be satisfactorily follow the frequency distribution behaviour of all clustered stations in each region.

Table 2. Summary of accuracy of distribution functions in the three homogenous regions

<table>
<thead>
<tr>
<th>Region Dist.</th>
<th>F = RT (years) =</th>
<th>0.5</th>
<th>0.8</th>
<th>0.9</th>
<th>0.95</th>
<th>0.98</th>
<th>0.99</th>
<th>0.995</th>
<th>0.998</th>
<th>0.999</th>
</tr>
</thead>
<tbody>
<tr>
<td>I GEV</td>
<td>q(F)</td>
<td>0.979</td>
<td>1.203</td>
<td>1.332</td>
<td>1.444</td>
<td>1.573</td>
<td>1.659</td>
<td>1.737</td>
<td>1.828</td>
<td>1.889</td>
</tr>
<tr>
<td></td>
<td>B^2(F)</td>
<td>0.000</td>
<td>0.000</td>
<td>0.001</td>
<td>0.001</td>
<td>0.003</td>
<td>0.004</td>
<td>0.005</td>
<td>0.007</td>
<td>0.009</td>
</tr>
<tr>
<td></td>
<td>A^2(F)</td>
<td>0.044</td>
<td>0.050</td>
<td>0.056</td>
<td>0.063</td>
<td>0.072</td>
<td>0.079</td>
<td>0.085</td>
<td>0.094</td>
<td>0.100</td>
</tr>
<tr>
<td></td>
<td>R^2(F)</td>
<td>0.019</td>
<td>0.021</td>
<td>0.024</td>
<td>0.026</td>
<td>0.030</td>
<td>0.033</td>
<td>0.035</td>
<td>0.039</td>
<td>0.042</td>
</tr>
<tr>
<td>II GEV</td>
<td>q(F)</td>
<td>0.990</td>
<td>1.217</td>
<td>1.339</td>
<td>1.439</td>
<td>1.545</td>
<td>1.612</td>
<td>1.669</td>
<td>1.732</td>
<td>1.771</td>
</tr>
<tr>
<td></td>
<td>B^2(F)</td>
<td>0.002</td>
<td>0.000</td>
<td>0.001</td>
<td>0.000</td>
<td>0.003</td>
<td>0.006</td>
<td>0.010</td>
<td>0.016</td>
<td>0.022</td>
</tr>
<tr>
<td></td>
<td>A^2(F)</td>
<td>0.048</td>
<td>0.050</td>
<td>0.054</td>
<td>0.060</td>
<td>0.072</td>
<td>0.082</td>
<td>0.093</td>
<td>0.108</td>
<td>0.120</td>
</tr>
<tr>
<td></td>
<td>R^2(F)</td>
<td>0.043</td>
<td>0.044</td>
<td>0.048</td>
<td>0.054</td>
<td>0.064</td>
<td>0.073</td>
<td>0.084</td>
<td>0.099</td>
<td>0.112</td>
</tr>
<tr>
<td>III PE3</td>
<td>q(F)</td>
<td>0.960</td>
<td>1.234</td>
<td>1.401</td>
<td>1.552</td>
<td>1.736</td>
<td>1.868</td>
<td>1.994</td>
<td>2.156</td>
<td>2.275</td>
</tr>
<tr>
<td></td>
<td>B^2(F)</td>
<td>0.002</td>
<td>0.002</td>
<td>0.002</td>
<td>0.002</td>
<td>0.003</td>
<td>0.003</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
</tr>
<tr>
<td></td>
<td>A^2(F)</td>
<td>0.054</td>
<td>0.060</td>
<td>0.069</td>
<td>0.077</td>
<td>0.086</td>
<td>0.093</td>
<td>0.098</td>
<td>0.104</td>
<td>0.108</td>
</tr>
<tr>
<td></td>
<td>R^2(F)</td>
<td>0.015</td>
<td>0.017</td>
<td>0.019</td>
<td>0.022</td>
<td>0.024</td>
<td>0.026</td>
<td>0.027</td>
<td>0.029</td>
<td>0.030</td>
</tr>
</tbody>
</table>

4.2. Estimation of Uncertainty Analysis of Regional Quantiles

The Monte Carlo simulation procedure for determining the rainfall amounts for different return periods for each site in the regions were estimated and the measure for uncertainty was expressed using coefficient of variation. The comparison plot of coefficient of variation of regional and at-site analysis results are given in Figures 3 and 4. The GEV distribution was selected as at-site frequency distribution for the whole region in accordance with the studies in [6] based on L-Moments for yearly extreme rainfall. The standard deviation and mean of the quantiles of 1000 simulations for each return period is calculated and the parameter stability of the distributions is assessed using Cv. For return period 10 and 20 years as seen in Figure 3, coefficient of variation is seen to be nearly at same value for regional estimates in all
regions I, II and III, while at-site estimated quantiles are seen to fluctuate with a relatively higher dispersion in regions I and III. The sample size generated in each iteration is equal to the original sample size of 20 for each site. The at-site quantiles from Figure 3 shows that some sites in the at-site analysis have produced better estimates than regional analysis with a lower $C_v$, while the others were less accurate. For region I, despite both at-site and regional fit GEV distribution, parameters of regional GEV distribution seem to be more stable and reliable. The desirable outcome is to have $C_v$ of all regional estimates lower than at-site, and the probable reason may be due to small sample size of 20 considered in the study. But overall, the Figures 3 and 4 are suggestive of the fact that rainfall depths estimated on regional analysis produce more accurate and reliable estimations. The estimations in regional analysis for region II is seen to be quite similar to at-site estimations for all return periods. The region comprises of only two stations and so the prediction accuracy may not be fully represented for the region. The coefficient of variation is also seen to increase with return period for all regions for both regional and at-site estimations, thereby suggesting increase in uncertainty. But the at-site estimations increase at a higher rate. For example, region I at-site lowest and highest $C_v$ values increased from 0.039 and 0.109 to 0.085 and 0.365 respectively; whereas the increase for corresponding regional values are 0.064 and 0.070 to 0.162 and 0.171 respectively. Comparatively, region III gave very large variations for at-site estimations in all return periods. The results thus indicates that the regional estimate of quantiles is much more reliable and accurate compared to the at-site estimations.

Figure 3. Uncertainty of extreme rainfall estimations in homogenous rainfall regions for (a) 10 and (b) 20 years return period
Confidence Interval based Uncertainty Analysis

To test the confidence intervals for each region, two sites were considered – the lowest and the highest discordant stations in each region – to see the effect on these two extremities. The lowest and highest discordant stations in region I are Dibrugarh and North Lakhimpur; in region II they are Cherrapunjee and Mawsynram, while in region III they are Silchar and Kailasahar. Region II consists of only two stations, and the assessment is made only between them. The Kolmogorov-Smirnov (KS) and Anderson-Darling (AD) goodness-of-fit tests are used to examine how well the Empirical Cumulative Distribution Function (ECDF) and theoretical CDF fit the observed data. From the results of the goodness-of-fit test in Table 3, estimates for at-site and regional analysis for each region are explored, and the p-value results indicate that the observed data seems to come from a population with a PE3 distribution for region II and a GEV distribution for regions I and II. The p-values obtained further suggest that the highest discordant sites in each region seem to have less fit in comparison to the lowest discordancy sites when fitted through either at-site or regional analysis. For region II, the relative difference is not clearly distinguishable, but the p-values indicate a good fit in both sites.
The confidence intervals evaluated to estimate the uncertainty for the lowest and highest discordant stations in each region is presented in Figures 5 to 7. For Dibrugarh station in region I, the empirically determined precipitation quantiles all fall within the 95% confidence interval (CI) bounds for both MCS and normality assumptions. Highest discordant station of region I i.e., North Lakhimpur in Figure 5 shows wider CI bounds for at-site analysis with the empirical quantiles lying on the lower CI bounds of normality assumption. Thus, the 95% CI bounds are found less narrow and the uncertainty associated with estimation of at-site quantiles are relatively higher. In this region both the regional growth curve and at-site growth curve are well within the MCS CI and normality-based confidence intervals. For Cherrapunjee station in Figure 6, the upper limit of the CI is found to be almost similar in both CI bounds and do not show any significant difference. Thus, the uncertainty in quantiles for Mawsynram is least in region II. For station Kailasahar of region III in Figure 7, the confidence interval width was almost similar in both regional and at-site estimations. For Silchar with lowest discordancy in region III, the upper limits of the CI from regional approach have higher values thus providing greater widths for higher return periods. Thus, the uncertainty is seen to be more for lowest discordant station in this region. Though the empirical quantiles up to 8-year return periods are seen to fall on the lower limit of both the CI bounds, the observation circles are seen to be on the lower limit of both the CI bounds of both MCS and normality CI bounds afterwards.

The regional estimates remain close to the lower limit of the CI bounds of both MCS simulation and normality bounds, thereby indicating no overestimation in the quantile growth curve. Figure 5 to 7 overall indicates that the confidence intervals for regional quantiles calculated from MC simulation are narrower and follow normality assumptions in all three homogenous regions. The uncertainty is explored and presented for the discordant extremes in the regions. In comparison to at-site analysis, the regional approach was found to have low uncertainty in most of the stations.

To have an assessment of overall performance including all stations in a region, the average relative width (ARW) of the confidence intervals has been analysed and presented in Table 4. The ARW is calculated for other stations in the region and confidence intervals are computed at each return period. Comparison to at-site estimates and confidence intervals for corresponding return periods are also done. The results show that, the regional analysis of rainfall estimates for region I produced narrower confidence intervals than at-site analysis. For region II, the widths of CI’s across all stations in the region were slightly larger than at-site analysis CI’s. But the CI’s obtained for region II were relatively the least deviating among the three regions, thereby indicating that the GEV distribution satisfactorily describes the rainfall distribution in the region. For region III, the confidence intervals were better than those of at-site only for higher return periods of 500 and 1000 years. This indicates that the regional PE3 distribution was relatively less appropriate to at-site GEV distribution for the region III in producing accurate rainfall estimates. The clustered region III constitutes twenty stations which may be considered large, and the analysis has overestimation of regional growth curve from true at-site growth curve for some stations and underestimation on some stations. This may be due to the widespread location of the stations in both Brahmaputra and Barak basin.

### Table 3. Goodness-of-fit tests of lowest and highest discordant stations

<table>
<thead>
<tr>
<th>Homogenous Region</th>
<th>Site</th>
<th>Analysis Method</th>
<th>AD Test Statistic</th>
<th>P-value</th>
<th>KS Test Statistic</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>At Site</td>
<td>Dibrugarh</td>
<td>0.273</td>
<td>0.976</td>
<td>0.150</td>
<td>0.983</td>
</tr>
<tr>
<td></td>
<td>Regional</td>
<td></td>
<td>0.332</td>
<td>0.937</td>
<td>0.150</td>
<td>0.983</td>
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<tr>
<td></td>
<td>At Site</td>
<td>North Lakhimpur</td>
<td>0.373</td>
<td>0.902</td>
<td>0.150</td>
<td>0.983</td>
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<td></td>
<td>Regional</td>
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<td>0.552</td>
<td>0.715</td>
<td>0.200</td>
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</tr>
<tr>
<td>II</td>
<td>At Site</td>
<td>Cherrapunjee</td>
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<td>1.000</td>
<td>0.100</td>
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<tr>
<td></td>
<td>Regional</td>
<td></td>
<td>0.229</td>
<td>0.993</td>
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<tr>
<td></td>
<td>At Site</td>
<td>Mawsynram</td>
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<td>0.998</td>
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<td>0.213</td>
<td>0.996</td>
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</tr>
<tr>
<td>III</td>
<td>At Site</td>
<td>Silchar</td>
<td>0.206</td>
<td>0.997</td>
<td>0.100</td>
<td>1.000</td>
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<tr>
<td></td>
<td>Regional</td>
<td></td>
<td>0.194</td>
<td>0.998</td>
<td>0.100</td>
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<tr>
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<td>At Site</td>
<td>Kailasahar</td>
<td>0.187</td>
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<th>P-value</th>
<th>KS Test Statistic</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
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</tbody>
</table>
Figure 5. Comparison plot of confidence intervals for (a) lowest and (b) highest discordant stations for region I

Figure 6. Comparison plot of confidence intervals for (a) lowest and (b) highest discordant stations for region II
Furthermore, altitude appears to have an impact on the quantile performance among the stations in the cluster regions. The region I, II and III has altitudinal difference between stations with highest and lowest altitude as 56, 88 and 1582 m. Thus, it can be seen that as the altitudinal variation in a cluster group increases, there is seen to observe a reduction in the efficiency of regional quantile estimates. Region III has the largest number of stations and constitutes the highest and lowest station altitudes in the study area, and hence rainfall estimates is found to vary in the region relatively more, leading to more uncertainty. Thus, the results indicate estimates from regional analysis is most accurate in region I, with slightly reduced performance in regions II and better performance for only higher return periods in region III. The uncertainty in regional analysis estimates is thus explored and with comparison to at-site approach in the delineated homogenous regions, is considered preferable.

### Table 4. Average relative width (ARW) for regional and at-site rainfall quantiles

<table>
<thead>
<tr>
<th>Region</th>
<th>No of stations</th>
<th>Frequency Analysis method</th>
<th>ARW for different return periods</th>
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<tr>
<td></td>
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<tr>
<td>Region I</td>
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<td>Regional</td>
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<td></td>
<td></td>
<td>At-Site</td>
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<tr>
<td></td>
<td></td>
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<td>Region III</td>
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<td>Regional</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>At-Site</td>
<td>0.29</td>
</tr>
</tbody>
</table>

#### 4.4. Uncertainty Analysis based on Information Transfer Index and MLE-GA

For regional analysis, the parameters of the selected distribution determined using L-Moments in each homogenous region is taken for lowest and highest discordant sites in each region. For uncertainty determination of the parameters, ungauged site (lowest and highest discordant sites of a homogenous region) are considered for assessment. Information transfer index (ITI) values were calculated based on equation number 9 to 11, and new random samples at all the sites
in the homogenous region are generated using parameters of the selected regional distribution. Then, random sample data of the original sample size as the considered site is generated based on ITI dependent weights using Equation 12. The parameters of the ITI based random sample is evaluated using L-Moments method and rainfall depths under different return periods (T= 10, 20, 50 and 200 years) is determined. The process is repeated 1000 times using Monte Carlo simulation approach, and quantiles under different are obtained. For each return period, estimated rainfall quantiles from 1000 Monte Carlo simulation are sorted and ranked and the 5th, 50th and 95th percentile is obtained. Similar procedure is applied for the other two types of weights viz. (i) distance-based dependent weights, and (ii) combination of ITI and distance-based weights to generate random sample from regional distribution at the considered site. Here, the distance is based on Euclidean distance and weights are determined using equation 13 and 14. In the present study, the stations of region 2 was not considered in the analysis, as ITI and distance-based weights for only two stations was not possible. Two new stations Karimganj and Lengpui was considered in the study for assessing the performance of the analyses. The sites were assigned to region III based on the Euclidean distance nearness of station attributes to the centroid of region III cluster group.

4.4.1 Regional Uncertainty

The ITI based weights gave better results in regional frequency analysis with estimates of rainfall performing better than at-site analysis for both least and highest discordant stations in region I; and for least discordancy of region III. Despite the fact that the fitted distribution for region I is both GEV distribution in both regional and at-site frequency analysis, L-Moment based regional rainfall estimates was found to clearly outperform the at-site estimates for all return periods and can be seen in Figure 8. This shows that, stations with least and highest discordancy in the region I gives better prediction with regional frequency analysis compared to at-site frequency analysis. The uncertainty in quantile estimates for regional analysis was observed the lowest with ITI based weighting and highest for distance-based weighting in all return period of the two regions except for highly discordant station in region III. This may be due to high regional absolute bias A(F) of region III as presented in Table 2, as a higher value is suggestive of estimation of quantiles to be consistently high at some stations and low at others. The performance of the new method for two new stations i.e., Karimganj and Lengpui as in Figure 10, did not provide acceptable results as the regional estimates were significantly much higher compared to at-site estimates. One reason for this may be due to the data for the stations may not behave as the selected regional distribution for the homogenous group and may need to be included in clustering for proper allotment of homogenous group. But the performance of ITI based uncertainty compared to at-site was superior for Lengpui station, thereby suggesting the ITI based method of generating station data to be reliable and robust. Overall, for homogenous regions with low bias as in region I, the performance of ITI based uncertainty definitely outperformed at-site frequency analysis.

4.4.2 At-site Uncertainty

The ITI and distance-based weights for application of uncertainty in at-site frequency analysis was done considering 8 and 16 nearby surrounding stations. The grouping of stations into 8 and 16 stations was done by ranking and sorting the stations in terms of higher ITI value shared between the ungauged station (here least and highest discordant stations) for ITI dependent weights. For the distance-based and MLE-GA approach, the nearness to the study station of other stations was based on Euclidean distance. Figures 8 and 9 shows at-site estimates produced higher uncertainty in comparison to regional analysis estimates except for highest discordant station in region III. Applying the ITI based dependent weights, the uncertainty of at-site estimates was significantly reduced in comparisons to distance based at-site estimates. This result is suggestive of the fact that grouping of stations based on ITI yield much more reliable and correct information at the ungauged site. The uncertainty in rainfall estimates calculated by at-site estimates for all three approaches (ITI, distance-based and MLE-GA) is seen increasing with increase in return period for all stations, which is in agreement with the corresponding results of regional estimates. However, the rate of increase is reduced with inclusion of more extracted sites from 8 to 16. The uncertainty obtained for MLE-GA based estimates for both ITI and distance-based is found to have lower values compared to at-site ITI and distance-based estimates for return period of 50, 100 and 200 years for all regions. This suggests that the regional analysis comparison to at-sites estimates based on MLE method optimized by genetic algorithm estimates are more preferable. Though in many studies, it is generally observed that L-Moment method outperforms MLE method in regional frequency analysis, and MLE generally performs better with larger sample size. The present work found MLE to perform better than L-Moment method in at-site frequency analysis estimates and with low sample size of 20. For the two new sites Karimganj and Lengpui as presented in Figure 10, the MLE-GA method also performed better with least value of uncertainty for at-site estimates both for ITI and distance-based estimates. This suggests that the at-site estimates based on MLE-GA may serve as a better alternative for comparing regional frequency estimates.
Figure 8. Uncertainty of extreme rainfall estimations for (a) least and (b) highest discordant stations in region I.
Figure 9. Uncertainty of extreme rainfall estimations for (a) least and (b) highest discordant stations in region III

Figure 10. Uncertainty of extreme rainfall estimations for two ungauged stations (a) Karimganj (b) Lengpui not considered in grouping of homogenous regions
4.5. Comparison with Previously Done Similar Studies

Although this is the first study to present a comparison of regional and at-site rainfall estimates in the southern part of Brahmaputra and Barak region, some closely related studies in other parts of the world may be related for validation of the performance. For at-site analysis in Maryland, USA, Al Kazbaf and Bensi, 2021 [14] found that the choice of distribution and method of parameter estimation (LMOM, MLE and MOM) affected the shape and location of precipitation estimate hazard curve performance significantly. For regional analysis with GEV and GNO distributions, the effect of distribution choice had limited effects. This is in accordance with the results in the present study wherein GEV distribution of at-site analysis seems to perform better than regional analysis by PE3 distribution for stations in region III. While for region I where both at-site and regional analysis are based on GEV distribution the regional analysis performed better in uncertainty. So, the choice of distribution is an important parameter in regional frequency analysis. Also, the parameter estimation method MLE-GA was found to perform better for highly discordant site and lower to least discordant site in region III compared to regional analysis. Yin et al. 2016 [33] compared the accuracy of regional and at-site quantiles of Yangtze River delta region based on RMSE and obtained lower RMSE for regional analysis for longer return periods. Li et al. 2019 [15] considered the lowest and highest discordant stations in nine homogenous regions of Sichuan province, China and found that stations with lowest discordancy had smaller differences of design rainfall values for both regional and at-site frequency analysis compared to stations with highest discordancy in the region. This is seen in the study presented, with larger difference for highest discordancy sites in region I and III. Zhou et al. 2014 [12] compared the MLE and L-Moment method for annual extreme precipitation estimates in Taihu basin of China for GEV and PE3 distribution and found MLE to provide unreasonable higher estimates compared to L-Moment estimates. In the present study, MLE method optimised using GA gave reasonable estimates for at-site analysis using GEV distributions and performed better to at-site estimates based on L-Moments method. But the precipitation estimates based on L-Moment regional frequency analysis performed superior to at-site analysis for both L-Moment and MLE-GA estimation methods for most stations. The MLE-GA estimates for at-site analysis for the stations in the homogenous regions from both ITI and distance-based estimates was accurately estimated with observance of no unreasonable result.

5. Summary and Conclusions

The study focused on the performance of extreme rainfall quantiles for homogenous regions delineated by genetic algorithm-based clustering. Uncertainty and accuracy assessment was investigated for the selected frequency distributions of the derived homogenous regions. Two distributions GEV and PE3 were found to satisfactorily define the annual extreme rainfall behavior in the study area. Regional growth curves of GEV and PE3 distributions from regional frequency analysis gave minimal bias and least deviation in all three regions. The uncertainty associated with regional rainfall quantiles is then reported using coefficient of variation Cv, and is found to be consistent and fairly low for all considered stations in the regions. Whereas analysis from at-site quantiles for the stations were seen to be highly inconsistent and produced higher values of Cv with increase in return periods. Results obtained suggest consistency in uncertainty of rainfall estimates for regional analysis, with larger variation in at-site analysis.

Results of uncertainty for regional quantiles of lowest and highest discordant stations in all three delineated homogenous rainfall regions did not seem to differ distinctly, and were within the confidence limits of both Monte Carlo simulation and normality assumptions. Whereas the uncertainty of quantiles estimated from at-site analysis increased after return period of 100 years in regions I and III. Results also show the uncertainty associated with rainfall quantiles derived from Monte Carlo simulation to follow normal distribution, and hence the regional rainfall quantiles were satisfactorily accurate. Region III comprised of 20 stations and were widely spread across both Brahmaputra and Barak basin. Growth curve in this region gave higher absolute relative bias for return periods up to 200 years, and may be attributed to a large number of stations with distinctly varying altitudes. Further, altitude seems to have influence on regional frequency analysis in the northeast region. As the altitudinal variation of stations for a cluster group increased, reduction in the accuracy of estimated regional quantile estimates was observed.

An assessment of overall performance of a homogenous region with average relative width of confidence interval showed that regional analysis produced narrower confidence intervals than at-site analysis in region I. While the average relative width (ARW) of region II and III were not as good as region I and had slightly higher values for regional analysis. But, the regional estimates was found to be better at higher return periods of 500 and 100 years in region III. Based on the ARW results, genetic algorithm based clustering approach is found to be a robust method in determining homogeneous regions and hence can be applied in determining reliable and accurate rainfall estimates for any study region.

The ITI-based weights produced superior results in regional frequency analysis, with rainfall estimates outperforming on-site analysis for both the least and most discordant stations in region I, as well as the least discordant stations in region III. Except for the most discordant station in area III, the uncertainty was determined to be lowest when ITI weighting was applied and highest when distance weighting was utilized. Regional analysis outperformed at-
site frequency analysis based on ITI-dependent derived weights for homogeneous regions with little bias. For all regions, the uncertainty in quantile estimates using at-site analysis was consistently greater than the uncertainty in quantile estimates using a distance-based weighting technique. This finding suggests that ITI provides more valuable information across sites and can combine sites to provide more accurate information at an ungauged site. Additionally, the uncertainty associated with MLE-GA-based at-site estimates of ITI and distance-based estimates is shown to be lower and more preferred than that associated with at-site L-Moments-based ITI and distance-based estimates. While the MLE-GA approach performed best for the two new unmeasured sites Karimganj and Lengpui, it did so with the least uncertainty, indicating that at-site estimates based on the MLE-GA method may be a preferable choice for comparison. The results of this study will be helpful in promoting the differences between regional and at-site frequency estimates in the context of hydrological frequency analysis. At the same time, it will be helpful in assisting decisions related to risk and hazard mitigation of extreme rainfall events in the northeast region of India.

6. Declarations

6.1. Author Contributions

N.D. contributed to the conception, design and write-up of the manuscript; P.R. and P.C. guided and supervised the research work; S.A. reviewed and edited the first draft 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 on request from the corresponding author.

6.3. Funding

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6.4. Acknowledgements

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6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Urban Planning and Reconstruction of Cities Post-Wars by the Approach of Events and Response Images

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Abstract

The research attempts to shed light on how to invest the philosophical and intellectual concept of the event in preparing the development plans for the city. Based on it, there are three strategies to read the event (Explanation, Interpretation, and Deconstruction) that are regularly responded to it with three strategies represented by (Revitalization, Renewal, and Reform). Through the use of reading and response strategies, and the corresponding planning policies represented by: preservation, rehabilitation, and redevelopment. The research adopted an analytical and descriptive methodology for some world experiences for the eventful cities, such as Warsaw, which reflects (Explanation - Revitalization) and used preservation, Bilbao, which reflects (Interpretation - Renewal) and used rehabilitation, and Tianjin, which reflects (Deconstruction - Reform) and used redevelopment. In an attempt to benefit from these experiences and derive some indicators for each strategy. By applying the derived indicators to the traditional Mosul city, it concluded that the most appropriate strategy for the reconstruction of this city is the strategy of Explanation – Revitalization, which represents preservation because the destruction of the city was intending to crush the historical and cultural value of the city and destroy the local and national identity.

Keywords: Event; Response; Rehabilitation; Conservation; Redevelopment; Reconstruction.

1. Introduction

The urban structure of cities is affected by the factors that contributed mainly to its emergence through the moral and material aspects, where city planning is an expression of the individual’s relations with society, the material elements of that city, the natural environment, material values, customs, and traditions...etc. These interrelationships between man and aspects of his urban environment are a reflection of his civilization and the vastness of the human mind from a historical perspective, and the history, civilization, and culture that the city holds, which were formed by the various events that the city experienced. Legislation of international conferences and decisions related to the maintenance and preservation of urban heritage identified many aspects that should be taken into account when dealing with valuable historical buildings and facilities and according to multiple policies that differ according to the situation concerned with the preservation and the availability of different levels of preservation and protection of the value related to the origin of its various historical, social or cultural types or economic, etc. Here, the problem of choosing the type and method of dealing with the distinguished historical monuments affected in the Mosul city arises because each case is unique in its circumstances and the reality of a situation that may differ significantly from other cases, which calls for conducting appropriate studies for each of them [1, 2]. The Mosul city has gone through the

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Eventful cities can be defined as those that arise as a result of important historical events of religious or worldly importance or that involve them. The urban fabric of those cities that arose and developed according to successive stages and in complementary relationships with those manifestations and according to their importance, as they are considered the part of the whole urban previously and its continuity and development in the future [5]. Al-Samurai and Al-Qaraghuli (2021) dealt with the importance of the concept of sustainable development and the need to adopt it when reconstructing cities after the events of wars in particular. The descriptive approach was adopted in its study of previous experiences in the reconstruction process to reach a comprehensive and integrated approach to sustainable development that takes into account its various dimensions (economic, social, and environmental) as well as adding other dimensions such as (institutional, organizational, cultural) and adopting a set of principles, means, and techniques to reconstruct the Mosul city after the war according to a solid scientific approach [6].

Salman et al., (2021) discussed the morphological change in the urban system of cities after the events of wars as a result of the destruction inflicted on them and the importance of the reconstruction programs taking into account the preservation of the urban and architectural identity of the cities that were destroyed and vandalized due to the events of wars, taking into account the new situations after the wars and the need to highlight the events of the wars that passed through the city. As it has become part of its history, and the strategies used should take into account the integration of ancient sites that must be preserved and revived and integrated with the sites that resulted from the war [7].

Boloorani et al., (2021) dealt with the events of the ISIS war on Mosul city and its effects on the physical fabric and the social and economic situation. The study assumed that the method of re-evaluating damaged buildings after the destruction event plays an important role in resettling residents in their cities. The study relied on Synthetic Aperture Radar (SAR) techniques in evaluating and surveying damages to war-affected areas. The study found that 40% of the city's buildings were destroyed. The study was able to draw a comprehensive map of the sites of destruction, and prepare it as a tool that can be used in the reconstruction of the city [8].

Richards, G., (2020) suggested that the issue of urban events was previously neglected by researchers in urban studies. As many consider it to have a small and imperceptible impact on urban development processes. Recent studies have proven the opposite, as events have been added as a key factor in building urban development strategies for the regions, especially after the emergence of the concept of eventful cities. Event-based policies include many economic, social, and cultural indicators that can regulate and direct the growth of cities. The study relied on employing its idea by reviewing examples of the cities of Barcelona, Austin, and Hertogenbosch [9].

Biao He et al., (2020) pointed out the impact of mega-events as a strategic approach that would contribute to changing the city's image and increasing urban development. The study relied on a statistical analysis method to study the development in the Chinese city (Boao) in the year 2010-2015. The study proved the existence of a clear statistical relationship between urban development indicators and the events taking place in the city. The researchers found that international events can significantly enhance urban development in host cities. The study recommended that decision-makers have the responsibility to invest in events and employ them in directing urban development in an integrated manner to serve the interests of the people and the city [10].

In their study published in (2019), Shaimaa H. Hussein et al. tried to shed light on the event of destruction caused by the ISIS war on Mosul city. The study aims to develop a strategy for the development process through which the city can be reconstructed in a way that preserves its identity and architectural heritage. The study was able to put many indicators through which it is possible to ensure the redevelopment in a way that preserves the texture of the urban environment and social cohesion, restores collective memory, and achieves economic development. The study recommended the need to preserve the buildings of historical and symbolic value, as they have a major role in restoring the urban identity and enhancing the sense of belonging to the place [11].

Devine and Quinn, (2019) pointed to the impact of social capital in achieving sustainable societies. The study relied on qualitative studies and methods and in-depth interviews about the UK City of Culture 2013 (CoC13) event as
a social event that can contribute to achieving a sustainable city. The study found that this social event had a prominent role in achieving interaction and strengthening links between the social fabric, especially among the youth group of event organizers, as the event contributed to building trust, cooperation, and goodwill between organizers and volunteers. The study found that social events can enhance the dimensions of sustainability, achieve the social pillar and build social capital [12].

McGillivray et al., (2019) focused on the impact of the sporting event represented by the Olympic Games in Rio in 2016 and its impact on the urban development processes in the entire city to accommodate different sports and cultural activities. The study focused on how this event invested in accelerating the development of the city by creating commercial enclaves to maximize the benefits of the city. As tourists can be taken advantage of to enhance the economic potential. The study found the role of employing legal force that would create appropriate conditions for managing urban spaces and developing these spaces by directing visitor flows and knowing their consumption patterns and needs [13].

Previous studies dealt with multiple and important issues in the subject of events. Some of them studied the impact of sporting events or cultural events and their role in promoting urban development, and other studies focused on the impact of war events on cities because of the bloodshed left by wars and the great social, economic, and urban effects on cities. Some studies suggested the documentation process and the preparation of a comprehensive survey and documentation plans that would be a starting point for the reconstruction of the destroyed cities. Other studies adopted building strategies by formulating a set of indicators that could contribute to the renewal and reconstruction of cities.

From the foregoing, we note that there are limited strategies in dealing with the reconstruction of cities with the emergence of new situations and problems in cities represented by wars, floods, pollution, crises, and others. This generates the motivation to search for strategies and approaches that keep pace with these developments. Hence, the knowledge addition to the research came through deriving philosophical and intellectual concepts that support decision-makers in choosing the best approaches for urban planning and city reconstruction through reading events and images of response to these events. By explaining how to deal with different events and how to read each of them and choose the strategy that fits with the event and that achieves the most reasonable and realistic in the reconstruction. Our study differs from previous studies in its attempt to find a comprehensive strategy that can be employed in the issue of urban reconstruction by adopting philosophical and conceptual binaries that enable us to build a comprehensive and new base that would cover the spatial dimensions in the processes of redevelopment and reconstruction. These strategies are based on putting forward binaries in and can be interpreted according to Figure 1.

![Figure 1. Reading and responding to events](image)

2. Theoretical Approach

2.1. Strategies for the Development of Cities after the Events of Wars

A. Explanation (Revitalization): It can be defined as a conservation process used in significant areas of high quality for the function they perform, or of historical, cultural, and architectural value, where the buildings are maintained in good physical condition [14]. It does not mean preserving an entire area but may mean selecting a certain number of buildings to be preserved, either because they have historical or architectural value, and their condition or form is still good. A building may also have a religious, cultural, or social value, necessitating the preservation and restoration of the physical structure. This structure will interact with the urban fabric as a whole and thus will integrate with the social and economic aspects and remain in use within the urban fabric of the city [15].

Buildings are the reservoir of memories, so to violently and ruthlessly target and destroy these buildings is to target the memories of the residents. It is known that the identity of any cultural group is linked to the architecture and planning that represents them. Hence, any targeting of buildings and squares represents a targeting of the national identity, the targeting of which leads to the dissipation of feelings of belonging and collective memory, and feelings of separation from the place begin to appear. The strategy of targeting places that represent the national identity is used in
wars because it weakens the enthusiasm and morals among the population and thus facilitates their defeat by the enemy. Therefore, the most effective way to obliterate and defeat the identity of the community is to destroy its national identity by targeting buildings and places that bear historical and symbolic values [11]. As for places that do not bear special significance for collective memory and national identity, they are usually not targeted, because their demolition will not cause the desired emotional cultural shock, and therefore the main intended purpose of this type of war will not be achieved [16]. The places most targeted in wars and critically affecting identity are [11]:

1. Prepare and plan for the future by saving important documents and maps
2. Traditional buildings that carry deep values in the historical collective memory over the years, passed down by the residents from generation to generation.
3. Public buildings and places with symbolic values such as monuments, ancient forts, and city squares.
4. Buildings with distinctive architectural patterns and styles
5. Religious and cultural buildings.
6. Buildings that people know as service places or points of reference such as university buildings and schools.

**B. Interpretation or (Renewal):** is represented by the restoration of the building to perform its function again after carrying out the necessary maintenance or preparation to perform new functions or different activities that correspond to the spirituality of the building, its design, and architectural model. Rehabilitation is very important from an urban, economic and social point of view, and it can be adopted in areas where buildings have been partially damaged, or where there is an imbalance in the use of the land, such as the absence of green spaces, open spaces and lack of organization. It includes some improvements aimed at raising the efficiency of buildings and facilities, as well as removing their parts to provide some facilities and services that must be provided to the residents and the region [15]. Local urban development has been linked to the urban renewal of heritage sites and the work of the integrated concept of conservation and local community development as an innovative creative process aimed at preserving heritage, cultural and aesthetic aspects in addition to developing environmental, social, and economic aspects [16]. Renewal strategies aim to avoid the idea of static preservation and are not an attempt to “petrify” the past and turn it into a kind of open-air museum. Advocates of renewal policies stress the importance of a comprehensive and integrated approach to planning traditional areas, especially the need to consider full conservation/rehabilitation areas. Of course, certain buildings of special historical and/or architectural interest must be preserved as part of the overall scheme. But the real focus is on the activities and uses of the buildings as a whole, and the need to upgrade selectively and adaptively. This renewal approach raises a variety of critical issues [17]. Accordingly, five main indicators of the renewal strategy can be identified as follows [18]:

1. **Urbanization indicators**: include the following:
   - Adapting the historical quality of the mixed-use environment in line with contemporary conditions.
   - Preserving the urban pattern and textures of the historical city areas (of great importance) in the face of the necessary improvements and changes in land use.

2. **Economic indicators**: include the following:
   - Enhancing the contribution of the old region to the urban economy.
   - The economic role of tourism in the historical context.
   - The effect of the increased land value and/or taxes.
   - The efficiency of old land uses and new activities

3. **Social indicators** include the following:
   - Active participation of the poor, who generally constitute the majority of those living in historical areas, in the rehabilitation process.
   - Preserving low-income people in the face of changes in land use and value.
   - Protecting low-income populations from the impact of “optimization”.

4. **Cultural indicators** include the following:
   - Contribution of rehabilitation projects to strengthening indigenous cultural traditions and forms.
   - The role of historical city centers, their physical characteristics, and their social life in the local culture.
   - The importance of historical city centers as an area of special tourist interest.
5. **Political indicators**: Include the development of a national policy to support the participation of the urban heritage community in the formulation and implementation of renovation plans.

**C. Deconstruction (Reform)**: This method applies to destroyed areas or damaged buildings in very poor conditions, which includes removal and reconstruction of the land according to a new scheme that reflects the positive land use and population distribution pattern. The redevelopment process aims to remove polluted areas, reduce poverty and revitalize work areas in city centers. Making room for the expansion of vital institutions, such as expanding schools and others. Providing environmentally friendly industries and expelling polluting industries. Encouraging middle-income families to continue to live in city centers [15]. The focus on redevelopment allows for an increase in the quality of the internal environment, as well as in improving mobility and enhancing local economic activity [19]. Strengthening the local economy helps improve the quality of life for the residents and promotes interest in the area, either as a commercial and service center or as a central component of heritage and cultural tourism. Strategies such as demolition, evacuating public spaces, or increasing building heights can cause inconvenience and anxiety to residents. These strategies must be properly organized and coordinated to minimize their impacts, particularly those involving the generation of construction waste, increased density in the face of value created by the occupation of existing buildings, an increase in the size of new buildings, an increase in paved surfaces and the consequent reduction in green space that may be little presence in the area [20].

### 2.2. World Lessons in the Planning of Destroyed Cities after the Events

**A. City of Warsaw /Poland (Explanation = Revitalization)**: One of the most important planning examples about the war event and the systematic destruction of heritage and identity, and the resulting response represented by the neighborhoods, is what happened in Warsaw during the Second World War. The Germans systematically destroyed the city and its cultural heritage. Much of Warsaw was destroyed as a result of the war [21]. The Germans identified important monuments, memorials, and buildings of symbolic and historical value in addition to buildings with distinctive architectural styles and decorations, and then the Nazi forces destroyed these places, and they blew up legal places and set fire to all homes and streets one by one, and the result was: Demolition and destruction of more than 84% of the urban fabric in the city. This German policy (by destroying the city and the buildings in it because of its identity, history, and heritage) in Warsaw was a way to crush the spirit of resistance among the Polish people, shake the confidence of the Poles in themselves, strike identity and erase history [22].

![Figure 2. The destruction in Warsaw after World War II (destroyed areas - In red color)](image-url)
The city of Warsaw responded to this event by interpreting it as an attempt to erase and remove its identity. It adopted a strategy of revitalization by re-planning and building all that had been destroyed without significant change. Architects, planners, historians, educators, archaeologists, and all the intellectuals of the city’s people who appreciate the meaning of the city and identity, and who look forward and the future, took the initiative. Documentary plans for the historical city of Warsaw for fear of the German Nazis. After the end of the war, documents and plans were taken out, which were in good and intact condition, and were used as a basis for rebuilding the city between 1945-1966. This was done by preparing and planning for the future of the city and preparing a plan for reconstruction that guarantees the preservation of identity, the preservation of collective memory, and the disclosure of the oldest layers of history, with the help of all the institutions of society [23, 24].

Figure 3. The destruction that occurred in the Polish city of Warsaw after World War II and its reconstruction according to the revitalization strategy

B. City of Bilbao / Spain (Interpretation = Renewal): The development of the city of Bilbao came after the traditional city was subjected to an unprecedented flood disaster in the year 1983, which led to the destruction of infrastructure and distortion of urban landmarks and then the economic crisis that came after that in 1990 and hit the heavy industry base seriously, and the bankruptcy of many factories, which caused social problems also represented by the loss of job opportunities [25]. Then the phase of building a modern city began, focusing on developing the cultural sector, rebuilding the city and building a very advanced system for public transportation, in addition to building a large number of private museums and basic facilities in the cultural, educational and entertainment fields [26]. The response to the most prominent event in the transformation was by transforming its metal industry and its port. The Bilbao strategy also relied on updated elements such as urban marketing and the creation of infrastructure and cultural facilities to develop a positive image of the city and to welcome a new economy. This strategy, which is based on the urban project, has been successful, as the announcement of the project has more impact than the project itself [27]. See Figure 4.
Figure 4. Photos for the destruction that occurred in the Spanish city of Bilbao after flood disaster and its reconstruction according to the renewal strategy

C. Tianjin Eco-city, China: (Deconstruction= Reform): The Chinese city of Tianjin extends over an area of 30 Km² and was designed by Sorbana Group for Urban Planning to be a model for Chinese cities in the future. This city was chosen because it is the largest industrial base and commercial center in North China and to treat pollution-reading the event resulting from economic and commercial activity because this city is an important production and marketing base in northern China [28]. Where what was spoiled for the city’s environment was fixed-response to event-, as it adopted the strategy of environmental sustainability and made the city to be at the forefront of environmental cities. The process of responding to the event was successful, represented in integrating economic and social plans with the environmental dimension and giving it a leading role, also taking into account the adaptation and acceptance by the community of the city. Thus, the process of reading and responding was sound and achieved the desired goals [29, 30]. See Figure 5.

Figure 5. a) The pollution in the Chinese city of Tianjin; b) The response to it according to a reconstruction strategy designed by Sorbana Group for Urban Planning

The basic idea is a reading of the event, where the event contributes to the formulation of thought as much as it recognizes the facts and realities that are generated from it. Thought that denies the event turns into an illusion or a rigid belief. As for the thought that practices the event, it is an open possibility for new revealing explanations or innovative and fruitful readings. There are three strategies for reading the event: (explanation, interpretation, and deconstruction). There is overlap, coexistence, and attraction between them.

- Explanation gives priority to meaning over the text and the reader (or event), as the explainer claims that it reveals the author’s intention and the significance of the discourse, so it is based on analogy and imitation.

- Interpretation is the search for lost meaning and the rebuilding of intractable understanding. It is the strategy of the self to gather meaning by contrast.
Deconstruction gives priority to the text over the subject, meaning and reference, because the deconstructor does not care about what the text says, but rather pays attention to the discourse that hides itself and its truth, for this reason, this theory constitutes a strategy for the text-based on veiling and deception.

3. Research Methodology

The study methodology relied on the concept of the event in explaining the systematic destruction that Mosul city was subjected to (see Figure 6). The concept of the event included three pairs that were adopted as strategies in interpreting the reading and responding to the destruction event of Mosul city. The strategies are represented by (Explanation - Revitalization, Interpretation - Renewal, and Deconstruction- Reform). After that, conceptual interviews were found for these strategies in the field of urban planning and design (conservation, rehabilitation, and redevelopment).

![Figure 6. The proportions of destruction inflicted on the old city of Mosul](image)

Our study presented some examples to illustrate the application of different strategies. It was represented in the cities of Warsaw, Bilbao, and Tianjin, respectively, to prepare to enter into the analysis of the applied side (traditional Mosul city) according to the indicators derived in the theoretical side. As a result, the traditional city was severely damaged by the ISIS war, and nearly 15,000 buildings were destroyed, according to the initial assessments of the Al-Habitat Organization. A high percentage of these buildings were residential buildings, in addition to many historical sites and landmarks located on the west bank of the river, and the level of destruction has been described as unparalleled since the Second World War. (See Figure 7).

**Reading the event:** According to the map above, we find that 11% were destroyed and 26% sustained multiple damages, in addition to 63% of the buildings had major and minor damages [31]. Therefore, we find that the destruction targeted the entire traditional city as it represents the main center and is embodied in it. The historical and cultural dimension of the city in particular and Iraq in general, and this destruction was deliberate because the city is a mosaic of all religions, cultures, and nationalities. Therefore, the most appropriate reading of this event is the Explanation-Revitalization.

**Response to the event:** The response must be at the level of the damage caused to the city if we want to return it to its previous era, and since the reading was interpreted as that ISIS wanted to obliterate the cultural meaning of the city by destroying buildings that express its history and heritage and destroying its hundreds of years old signs, Therefore, the most appropriate strategy for this response is the revitalization.
4. Case Study

The Mosul city is the center of the Nineveh Governorate, located in the north of the Republic of Iraq. The Mosul city is located astronomically at the intersection of longitude 43.8° east and latitude 36.12° north, i.e. in the transitional zone between the plain and the mountains, where the city occupies a focus of polarization for the various lines of movement coming from different areas in their natural characteristics. The Tigris River is one of the most important geomorphological features, as it divides the city into two unequal halves, and the lands adjacent to the course of the river are one of the most attractive areas for housing and for various land uses [32]. The city is on both sides of the river, and the river's elevation is about 210 m at sea level. The surface structures of Mosul city are part of the topographical features of Nineveh Governorate, where the height of its surface features ranges between (360-220 m). Therefore, the surface of the city is generally characterized by more hilly structures than mountainous ones, and the western part of the city is generally characterized by being higher and undulating than the eastern part (except for some of the hills) in addition to containing many valleys that flow towards the east towards the Tigris [33].

The number of archaeological sites in Mosul, according to data from the Iraqi Ministry of Tourism and Antiquities, amounts to 1,791 archaeological sites, 250 heritage buildings, and 20 huge libraries containing important Islamic and Christian manuscripts. It was a target of ISIS, which sought to obliterate the city's identity by destroying its history, heritage, and archaeological landmarks, such as the Prophet Ayyub Mosque and the Al-Nuri Mosque, to strike its rooted position in history [34]. See figure 8. The city went through a series of systematic destruction after ISIS took control of it in 2014. As a result of the military operations that the city witnessed to liberate it from the organization, the city was subjected to massive destruction, reaching 80% in its right part, which represents the originality in its heritage, roots, and architecture, which is the real starting point for the restoration of Building the features of the traditional city as the original and important aspect inherited by the city, which must be preserved on the one hand and repurposed in the construction, reconstruction and development of the city as a whole on the other hand [35]. It is also known that the urban planning process for new cities is easier than re-planning the existing cities, so how if they have gone through crises represented by wars, pollution, or others [36]. The process of reconstruction of the Mosul city
after the event of the war must take into account the importance of the urban form that expresses the physical characteristics that constitute the urban areas as well as the shape, size, density, and composition of settlements, which can be viewed at different levels starting from the regional level, the urban, the neighborhood, and even the block and the street [37]. According to these circumstances, the historical dimension and authenticity of Mosul city must be preserved, and the reconstruction process should be following a rational approach after the events of the war [38]. These circumstances necessitate the adoption of the urban neighborhoods strategy in the traditional Mosul city, to be able to reduce the damage that is still inflicted in the historical and heritage buildings in the city, where they carry out various functional activities [31].

Figure 8. a) The location of Nineveh Governorate; b) The administrative units for the Nineveh Governorate and Mosul city location; c) The Built environment for the city center represented in of Mosul city

5. Results and Discussion

5.1. Prepare and Plan for the Future by Saving Important Documents and Maps

There are many maps, photographs, and documents of the traditional Mosul city, and by reviewing it, it becomes clear that the city consists of narrow alleys, organic compact fabric, traditional houses with wooden windows, and local building materials. This reflects the cultural heritage of this city ISIS has targeted all these important elements in the traditional city, and nearly 60% of the city's fabric has been destroyed [1]. The revitalization strategy of the traditional city in Mosul can be implemented by making use of all the master plans, maps, and aerial photographs that documented the heritage characteristics of the city to use them in the revitalization strategy as well as the efforts of the UN-Habitat and its experience in this strategy (See Figure 9).
5.2. Traditional Buildings that Hold Deep Values in the Historical Collective Memory Over the Years, Passed Down by Residents from Generation to Generation

The traditional city includes a large number of heritage houses that include the characteristics of traditional dwellings of Iraqi architecture, such as wooden doors and windows (Al-Shanashil), the opening towards the interior, and the deaf facades built of bricks [39]. What distinguishes their homes is their use of local materials, and traditional architectural methods inspired by ancient Iraqi architecture, such as the use of domes, vaults, and arches in roofing operations. In addition to these general advantages, there are special advantages in some of the alleys represented in linking some of the close-to-close houses with brick or wooden arches connecting them to consolidate the bonds of family cooperation [40]. Because of the war, many of these characteristics that affect the collective memory of the people of the city have been destroyed. Therefore, a revitalization strategy must be adopted to revive these traditional buildings through reconstruction and restoration. An example of this indicator is the Castle of Shatabia, an archaeological site in Mosul city associated with people's memory and identity for the city. The history of the castle dates back to the Atabeg era in the 12th century AD. Despite the historical symbolism of the city and its lack of reference to a religious or ethnic identity (See Figure 10).

Figure 9. Mosul city before and after the destruction event as a result of the war [1]

Figure 10. The castle of Bashatabia before and after the targeting and destruction by ISIS
5.3. Buildings and Public Places with Symbolic Values Such as Monuments, Ancient Castles, and City Squares

The traditional Mosul city contains many buildings that represent a great symbolic value for the city’s residents and represent a mainstay in their local and national identity. An example of these buildings is the Great Al-Nuri Mosque, built in the year 1172 A.D. which contains the hunchback minaret called Al-Hadba, and it was built of bricks and its height (67m) and width (17m) [41]. It bears great historical value and represents an Islamic symbol of the traditional Mosul city. ISIS blew up the mosque and Al-Hadba minaret to strike the spiritual and historical values of this city and undermine its identity. Therefore, it is necessary to reconstruct these buildings with the same characteristics and details and to preserve their previous form as it is to enhance the symbolic dimension of the city to help the community adapt and integrate with the new reality after ISIS (See Figure 11).

![Figure 11. Al-Nuri Mosque and Al-Hadba Minaret before and after the destruction and targeting operation by ISIS](image)

5.4. Religious and Cultural Buildings

The Mosul city has a variety of sects and religions, which gives it a diverse cultural, religious and social character that represents one of the most prominent pillars of the identity of this city. For this reason, ISIS has targeted many religious and cultural buildings that represent this diversity, and it is targeting spiritual, religious, and cultural values. Iraqi archaeologists have found the remains of a 2,600 A.D. palace of the Prophet Yunus after it was blown up by ISIS in Mosul city [42]. In addition, many Assyrian imperial relics were found after the terrorist bombing in the remains of the tomb of Prophet Joseph. The tomb is located on the top Tal Nabi Yunus, east of Mosul, and is one of the two hills that are part of the ancient city of Nineveh, dating back to the era of the Assyrian Empire. The terrorist organization ISIS dug deep tunnels under the ruins of the mausoleum to search for and steal artifacts, and these tunnels lead to an undiscovered palace dating back to 600 BC. In part of the tunnel, a scientist discovered Assyrian stone statues of demigods sprinkling holy water to protect the people under their care. In the same way, ISIS destroyed the Al-Tahira Chaldean Church, which is considered a masterpiece of architecture and art from the eighteenth century and one of the most beautiful churches in the East. And that the inner part of the church was relatively spared during ISIS occupation of the area between 2014-2017 and the upper part of the building was destroyed due to heavy bombing. On this basis, the strategy of revitalization the city's religious landmarks appears urgent to restore its diverse religious identity and the prospects for the peaceful coexistence of the various sects (See Figure 12).
5.5. Buildings with Distinctive Architectural Patterns and Styles

The Mosul city contains many buildings that represent a unique architectural style that contains many decorations on the walls and ceilings. Some of these decorations represent Islamic decoration and some of them contain geometric or plant motifs. These buildings have been greatly destroyed and lost their distinctive decorations, and therefore they must be reconstructed to preserve the aesthetic values of Mosul city. The most important characteristic of the buildings in Mosul city is their use of local materials such as Halan stone, alabaster, and plaster. The various plant and geometric decorations, in addition to animal and human carvings sometimes. In terms of the planning of these heritage houses, they are often close to the layout of the house, palace, and temple in the buildings of ancient Iraq, where the house yard alone constitutes a task overlooking the central courtyard rooms to ensure good ventilation and lighting. The buildings work to form a distinct and unique fabric and possess special features that give them an aesthetic and functional value, which leads to the fact that affluent living cities possess the richness and fecundity of character (See Figure 13).
5.6. Buildings that People Know as Service Places or Points of Reference, Such as University Buildings and Schools

Educational buildings represent one of the most important buildings on which people rely in their spatial memory, as service places that provide education and knowledge and as points of reference. Mosul city is full of many schools and educational buildings, some of which were established for long periods. As a result of the ISIS attack on this city and during the liberation battles, it was destroyed. Many of these buildings today require restoration or reconstruction. One of the most famous was Al-Sharqiya secondary school that was built in 1905 in the Islamic architectural style with arches and spaces, and a modern method of construction was used in its construction, which is called by the architects (Sandwich_Wall), meaning the insulating wall, and this method is based on the construction of two walls separated by an insulator consisting of a different material [43]. This style of building that the arches and the inner courtyard surrounded by rows have given a kind of simplicity and uncomplicatedness, which is positively reflected on the psyche of the students and pushes them to study and receive knowledge without any problem. The Sharqiya secondary building in Mosul city is linked to the minds of the Mosulis as an important chapter of education in Iraq. Therefore, it represents a moral and symbolic value based on which a revitalization strategy must be adopted to restore the moral values of the city (See Figure 14).

![Figure 14. Sharqiya secondary school in Mosul before, during and after its rehabilitation after the war against ISIS](image)

Table 1 shows the description of the indicates and the evaluation of the status of the urban structure for Mosul city.

<table>
<thead>
<tr>
<th>Indices</th>
<th>Description</th>
<th>Evaluation in the case study</th>
</tr>
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<tbody>
<tr>
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</tr>
<tr>
<td>Buildings and public places with symbolic values</td>
<td>The buildings that represent a great symbolic value for the city's residents and represent a mainstay in their local and national identity</td>
<td>The Great Al-Nuri Mosque, built in the year 1172 A.D. which is called Al-Hadba. It bears great historical value and represents an Islamic symbol</td>
</tr>
<tr>
<td>Religious and cultural buildings</td>
<td>Revitalization of the city's religious landmarks appears urgent to restore its diverse religious identity and the prospects for the peaceful coexistence of the various sects</td>
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</tr>
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</table>
6. Conclusions

The event is one of the tools of urban policy or a driver of the development process as a result of what it causes to stimulate urban development processes, and the possibility of its inclusion on the means that attract attention to urban areas, as well as the possibility of giving distinction to cities as a result of their various events. The war event is one of the major events that have repercussions on the city's planning, whether on the urban fabric or land uses distribution.

The reading of the war event varies according to its causes and results, and ISIS has deliberately targeted antiquities and heritage intending to obliterate identity, and therefore the reading relied on the strategy (Explanation - Revitalization) to preserve those areas and buildings and thus preserve its identity and the spirit of its community. The reconstruction process in this strategy requires the government to join forces with civil society organizations with a high awareness of the people.

It should be noted that the three strategies for the reconstruction of cities can coincide with defining the leading strategy according to the most influential event, as it is not limited to one strategy. We can focus the (Explanation - Revitalization) strategy in the traditional Mosul city, and the neighborhoods surrounding the traditional city can be developed by introducing updates to keep pace with modern developments through the interpretation-renovation strategy, as well as the possibility of establishing new sustainable neighborhoods through the dismantling-reform strategy to accommodate population growth. Thus, the integration of the three strategies of the Mosul city as a whole is achieved.

Determining the indicators of the three strategies allows urban planners and designers to deal with the event and respond to it with a new vision that enables to identify the major flaws that the event may cause, such as (the war event), to intensify efforts towards it, and to determine the appropriate planning policy. It also opens the horizons of research to explore the relationship between the event and the urban form, as well as the effect of the event on the spatial perception of people.

It is necessary to develop a strategy based on the principle of revitalization the urban, architectural, and heritage of the historic Mosul city through building legislation and governing conservation laws with the assistance of international expertise represented by UNESCO and the UN-Habitat, and that the rehabilitation of the historic area of Mosul requires preserving the architectural identity by preserving The collective memory and the urban character represented by the compact urban fabric, the nature of the land uses, the height of the buildings and the skyline, as well as the urban signs and symbols of the city - the main axes and nodes of the movement, as well as the architectural patterns and forms and decorations, thus preserving the moral and material values of the architectural and urban heritage.

7. Declarations

7.1. Author Contributions


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. Conflicts of Interest

The authors declare no conflict of interest.

8. References


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Effect of Masonry Infill Panels on the Seismic Response of Reinforced Concrete Frame Structures

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Abstract

The present work concerns the numerical investigation of reinforced concrete frame buildings containing masonry infill panel under seismic loading that are widely used even in high seismicity areas. In seismic zones, these frames with masonry infill panels are generally considered as higher earthquake risk buildings. As a result there is a growing need to evaluate their level of seismic performance. The numerical modelling of infilled frames structures is a complex task, as they exhibit highly nonlinear inelastic behaviour, due to the interaction of the masonry infill panel and the surrounding frame. The available modelling approaches for masonry infill can be grouped into two principal types; Micro models and Macro models. A two dimensional model of the structure is used to carry out non-linear static analysis. Beams and columns are modelled as non-linear with lumped plasticity where the hinges are concentrated at both ends of the beams and the columns. This study is based on structures with design and detailing characteristics typical of Algerian construction model. In this regard, a non-linear pushover analysis has been conducted on three considered structures, of two, four and eight stories. Each structure is analysed as a bare frame and with two different infill configurations (totally infilled, and partially infilled). The main results that can be obtained from a pushover analysis are the capacity curves and the distribution of plastic hinges in structures. The addition of infill walls results in an increase in both the rigidity and strength of the structures. The results indicate that the presence of non-structural masonry infills can significantly modify the seismic response of reinforced concrete "frames". The initial rigidity and strength of the fully filled frame are considerably improved and the patterns of the hinges are influenced by structural elements type depending on the dynamic characteristics of the structures.

Keywords: Reinforced Concrete Frames; Masonry Infill; Panels; Pushover Analysis; Plastic Hinges.

1. Introduction

Interior partitions and exterior masonry walls used as an infill between the beams and columns of a reinforced concrete framing are generally considered non-structural elements in design despite the experience of past earthquakes and events. Test results suggest that masonry infill generally exhibits an important influence on the seismic response of reinforced frame buildings. Reinforced concrete frame buildings with masonry infill panels in seismic areas are generally considered to be seismic risk buildings. As a result there is a growing need to evaluate their level of seismic performance. In most seismic codes, it is assumed that the infill has only influence on the mass of the composite structure. This would be a good hypothesis if the frame and the infill are well separated by providing a sufficient gap between them; however, in practice gaps are not usually specified.

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Over the past decades, much work has been devoted to the experimental study of this complex behaviour. Sasani (2008) [1] had conducted on site tests to study the dynamic response of a reinforced concrete building made with masonry infill brick panels revealing sudden damages in columns. The brick wall was modelled by a shell or equivalent compression strut members and the simulation results were compared. For conventional reinforced concrete buildings, brick infill panels are generally adopted for interior partitions. Negro & Colombo (1997) [2] noted that the partially filled wall can create a short column effect. The study by Mociran & Cobirzian (2021) [3] designed to verify the effect of the mechanical properties of masonry infill materials used in reinforced concrete frame structures, as well as the seismic performance of buildings.

Syrmakezis & Asteris (2001) [4] focused on the seismic performance of partially filled multi-storey reinforced concrete frames, using the contact point method to analyze the filled masonry frame and the effect of masonry panels with openings on variation of the stiffness of the filled frames. Perera (2005) [5] studied the performance evaluation of reinforced concrete structures filled with masonry under cyclic loading based on damage mechanics. Proposal of a damage model to characterize masonry walls subjected to lateral cyclic loading. The model includes simulations of phenomena such as the degradation of stiffness, strength and pinching behavior. The macro model is integrated into the nonlinear structural analysis program to analyze the reinforced concrete frame filled with masonry

Polyakov (1960) [6] proposed to first place the concept of diagonal spacer for the analysis of filled frames. (Tsai & Huang (2009) [7] carried out a study on masonry infill panels composed of compression struts to clarify their influence on the possibility of progressive collapse of a reinforced concrete building resistant to earthquakes. Syrmakezis & Asteris (2001) [4] have contributed that the main objective is to establish the relationships between the parameters of a wall opening (such as the position and the percentage of opening), as well as the study of the redistribution of action-effects (shear force diagram) of filled plane frames under earthquake loads. Zine et al. (2007) and Mao et al. (2008) and Kadid & Boumrkik (2008) [8-10]; concluded that the step-by-step pushover analysis procedure shows the performance level of building components as well as the maximum shear load capacity of the structure.

Crisafulli and Carr (2007) [11] noted that the resistance of a frame filled with masonry depends on the interaction between these two elements and that it is possible to modify their load resistance mechanism and their failure model. The research work of Rai (2009) [12] contributed to the idea that many traditional methods have been used to strengthen reinforced concrete structures such as the addition of reinforced concrete infill walls, prefabricated panels, steel bracing and concrete cladding element of the steel frame. Negro & Colombo (1997) and Mehrabi et al. (1996) [2, 13] noted that different infill frame damage mechanisms are possible, depending on the relative stiffness, the resistance of the filling and the delimiting frame border. Lourenço et al. (1998) [14] noted that strength and stiffness are greatly affected by infills. Maheri & Akbari (2003) [15] considered that the impact of the applied load and the type of support frame have a logically small effect on the estimates. Dautaj et al. (2018) and Villaverde (1997) [16, 17] concluded that over the past decades masonry infill walls have been used extensively in residential reinforced concrete frame structures, but their seismic effects are weak.

Aliari & Memari (2005) and Liu & Manesh (2013) [18, 19] noted that opening in frames filled with masonry reduces the strength, stiffness and energy dissipation of these frames. Koutromanos et al. (2011) [20] noted that significant damage to the filled wall creates a risk of falling. Benavent-Climent et al. (2018) [21] noted that the proximity of the infill dividers can adjust the global seismic conduct of the encircled structures and further modify the removals and the base shear of the edge. Inel & Ozmen (2008) [22] saw that delicate story due to padding dividers could be as damaging as a delicate story due to the increased height of the story.

The study by Shafaei et al. (2014) [23] evaluated the effects of flexible joints on the lateral response of reinforced concrete frames. Rahem et al. (2021) [24] have come to say that a number of static and nonlinear dynamic analyzes of the tensile history have been performed to assess the seismic vulnerability. Noui et al. (2019) [25] were able to assess the infill behaviour of reinforced concrete frames, focusing on the impact of infill openings of three types of reinforced concrete structures: rigid, semi-rigid and flexible, which were designed according to the Algerian seismic code, after a pushover analysis was carried out. They believe that the main parameters are related to the size, location and aspect ratio of the opening.

The study conducted by Mondal et al. (2013) [26] evaluated the behaviour factor of reinforced concrete structures of two, four, eight and twelve storeys. The objective of their research is to estimate the actual values of the response reduction / modification factor (R) of actual reinforced concrete frame buildings, which are designed, detailed in accordance with Indian reinforced concrete seismic design standards, while comparing these values with those suggested in the design code. Ricci et al. (2013) [27] reported that filled masonry walls are supplied in reinforced concrete frames worldwide for insulation against temperature, humidity, noise and fire [1]. Tavakoli & Akbarpoor (2014) [28], according to their research which focuses attention on the seismic performance and shear resistance of reinforced concrete frames filled with bricks under various lateral load models. This evaluation is carried out by nonlinear static analysis.
Seismic performances of reinforced concrete frames with masonry infill slabs under cyclic loading were studied by Jiang et al. (2015) [29]. They examined the impact of the construction details of the infill wall on the seismic performance of the reinforced concrete frame. They noticed that after adding masonry infill walls to the frame, the lateral strength, rigidity, and energy dissipation capacity of the exposed reinforced concrete frame were significantly improved, while the displacement ductility ratio was significantly reduced in the laboratory of the University of Bucharest, the seismic response of a reinforced concrete frame with masonry infill slabs was verified by Bolea (2016) [30]. Dautaj et al. (2019) [31] noticed that if separation joints are not provided between the walls and the frames filled with masonry, under seismic excitations, these walls could contribute to the mechanism of resistance to loads and to the fracture pattern of the frames in reinforced concrete constructions. Made the condition that if separation joints are not provided between the walls and the fill frames, under seismic excitations, these walls could contribute to the load resistance mechanism and the failure pattern of reinforced concrete frames.

Bamdad et al. (2016) [32], have studied the impact behaviour of solids and structures. Two common methods are the finite elements and the experimental method. The nonlinear finite element method has become one of the most efficient methods for predicting the behaviour and extreme strength, elasticity and strength of reinforced concrete beams from load pitch to failure. The nonlinear finite element method is one of the most effective methods for predicting the behaviour of reinforced concrete beams from load pitch to failure and its ultimate strength, elasticity and strength. The advantage of this method is its ability to make this prediction for all sections of the evaluated reinforced concrete beam and all loading stages. It is noted that the absence of infill walls at the bottom of the mediating floor will result in the formation of a soft floor mechanism [33]. Ahiwale et al. (2020) [34] studied open-story structures, the ground floor is exposed and the upper floors are filled with bricks. Have examined these structures and focused their exposure during an earthquake by demonstrating them as bare frames and as open story frames. Such a review was done to separate the presentation of the open story frame and the bare frame. (Vahidi and Moradi. 2019) [35]; (Feenstra and de Borst. 1993) [36] reported that at the lower level of the gallery, the masonry infill frame behaves like a monolithic composite wall; at higher drift levels (when the infill wall is separated from the boundary frame), compressive contact stresses will occur between the frame and the wall.

Ahiwale et al. (2020) [34] conducted a study to obtain the reaction of an open-story reinforced concrete building exposed to seismic tremor loads. Considered twelve storey structures on an inclined floor (150 horizontally). The reaction of the structure is evaluated in SAP 2000 software using a performance-based seismic design. Rahem et al. (2021) [24] opted for the ADINA software to verify the influence of the opening zone and the configuration of the window on the seismic performance of the steel frame. Non-linear digital static analysis and non-linear time history analysis are performed. Ahiwale et al. (2020) [34] research work ported on the evaluation of the seismic response of the existing reinforced concrete structure with an open history employing the design based on seismic performance. They proposed to think about the modernization of an open story using a reinforced concrete shear wall, steel bracing and infill wall.

Umar et al. (2020) [37] studied the effect of openings in infill walls on the performance of filled reinforced concrete frames, in other words, this research studies the number of infill walls in infill walls filled reinforced concrete frames. Rahem et al. (2021) [24] specified that the aim of the study is to examine the role of masonry infill on the damage response of steel framing without and with different types of opening systems subjected to nonlinear static analysis and to a nonlinear temporal analysis. Having studied, a complete evaluation is carried out using twelve types of steel framing without masonry, with complete masonry and with different heights and widths of openings. Hence the main objective is to present a seismic assessment framework approach for undamaged and damaged reinforced concrete structures. Umar et al. (2020) [37] studied two samples which were tested with reverse cyclic loading (quasi-static test). Also, they found, during experimental tests on the filled reinforced concrete frame having less opening in the infill wall, that this frame has more resistance to lateral loads, more rigidity and dissipated more energy compared to the frame having a large opening in the infill wall. Likewise, the displacement ductility ($\mu_D$) and the response modification factor (R) also depend on the amount of opening in the infill wall of a reinforced concrete frame.

Tiedeman (1980) [38] concluded that about 80% of the cost of structural damage caused by earthquakes is due to damage to infill walls and consequential damage to doors, windows, electrical installations and hydraulic equipment. Ahiwale et al. (2020) [34] concluded that there are plastic deformation formations at the level of the ground floor column. To counter the total collapse of soft story structures, it is necessary to modernize open story. Therefore, alternative measures are recommended to improve the reaction of the soft floor like reinforced concrete shear wall, steel bracing and infill wall.

Dya & Oretaa. (2015) [39] have come to say that the seismic demand resides at the level of the soft phase, and different are the levels of severity of this same phase which is manifested from the Pushover analysis used in the preliminary risk assessment tool. Jiang et al. (2015) [29] were able to conclude that reinforced concrete frames with masonry infill walls are widely used in buildings. The layout and construction details of infill walls have significant impact on the seismic performance of the reinforced concrete frame.
Other works have been devoted to the experimental study of this complex behaviour such as Smith (1962), Mainstone (1971), Klingner & Bertero (1978), and Flanagan & Bennett (1999) [40-43] analytically and numerically using the finite element method by Dhanasekar & Page (1986) and Stavridis & Shing (2010) [44, 45]. The strong influence of the mechanical properties of the materials used for masonry infills, on the seismic performance of reinforced concrete frame structures with masonry infill panels, located in different seismic zones, requires careful study of the seismic response to such structures.

In seismic areas, reinforced concrete frame buildings with interior partitions and exterior masonry walls, used as infill, are generally considered to be non-structural elements in the design despite the experience of past earthquakes and the results of past earthquakes. Tests suggest that masonry infill generally has a significant influence on the seismic response of such buildings. In this regard, there is a growing need to assess their level of seismic performance. The research flow chart is shown in Figure 1.

2. Modelling Aspects

2.1. Modelling of Infills

Numerical modelling of filled frames is a complex task, as these structures exhibit highly nonlinear inelastic behaviour, due to the interaction of the masonry infill panel and the surrounding frame. In general, the presence of the masonry infill panel and the interaction with the concrete frame changes the failure mechanism of the filled frame relative to the bare frame. To simulate the behaviour of the masonry wall, two available modelling approaches can be grouped into macro-models and micro-models. Macro-models, which attempt to capture the raw behaviour of the infill, by viewing the masonry as a homogeneous continuum with no distinction between individual units and approximate joints, are computationally efficient. On the other hand, micro-models capture the behaviour of the infill and its interaction with the frames in great detail, but these models are computationally expensive, because the masonry elements, the mortar, and the element interface masonry / mortar are modelled separately. A number of models using both approaches have been proposed by various researchers. In this study, the nonlinear layered shell element, implemented in SAP 2000 is used to model the infill panels. This approach allows any number of layers to be defined in the thickness direction, each of which has an independent material, thickness, behaviour, and location that may be non-linear.

Determining the basic mechanical properties of masonry is a very difficult task due to the large uncertainties. The dispersion of the measured values is very important. Kaushik et al. (2007) [46], conducted experimental studies on an analytical model to correctly plot stress-strain curves for masonry using six control points on the curves and resulted in a simplified tri-linear stress-strain model for masonry as shown in Figure 2.

2.2. Modelling of Reinforced Concrete Frame Elements

Nonlinear Static Analysis (Pushover Analysis) is a procedure presented and developed over the past three decades by many researchers. It is mainly based on the assumption that the response of the structure is controlled by the first mode or by the first modes of vibration, and that this shape remains constant throughout the elastic and inelastic responses of the structure. A two-dimensional model of the structure is used to perform a nonlinear static analysis, and the pattern of increasing lateral forces should be applied to the mass points of the system. The purpose of this is to represent all the forces that are produced when the system is subjected to seismic excitation.

2.3. Plastic Hinge Mechanisms

In this study, beams and columns, whose hinges are concentrated at their ends, are modelled as nonlinear with localized plasticity. SAP2000 implements the properties of plastic hinges described in FEMA 356. (2000) or ATC-40 (1996) [47, 48]. Figure 2, illustrates the five materialized points A, B, C, D and E which define the force-deformation, illustrated by SAP 2000, implementing the properties of plastic hinges described in FEMA 356. (2000) or ATC-40 (1996) [47, 48]. The following points should be targeted:

- Point A is continuously the origin;
- Point B represents the efficiency, whatever the value of the deformation specified for that point and the hinge up to point B will not be deformed. The displacement (rotation) of point B will be subtracted from the strain of points C, D and E. The single plastic strain beyond point B can be exposed by the hinge;
- Point C represents the ultimate analytical capacity;
- Point D indicates the analysis of the residual resistance;
- Point E defines the limit for a total failure.
The three dots: IO, LS and CP, used to define the acceptance criteria for hinges, represent immediate occupancy, personal safety and collapse prevention, respectively, and are defined by FEMA-356. SAP 2000 provides default hinge properties and recommends PMM hinges for columns and M3 hinges for beams. Once the steel content, the properties of the section and the loads acting on the structure are known, then default hinges are assigned to the elements. Thus, axial-flexible plastic hinges (PMM) are assigned to the ends of the columns while plastic bending hinges (M3) are assigned to the ends of the beams. The plastic hinges in the filled frames are concentrated in the lower levels of the structures, while in the bare frames the plastic hinges are distributed over the height of the structures, especially for the solid infills.

Figure 1. Flow chart of the study

Figure 2. Force-displacement curves of the hinges with colour codes
2.4. Structures Used

This study is based on structures with design features and details typical of Algerian construction modes. For the evaluation of the behaviour of the masonry infill of the reinforced concrete frames, provided by exterior walls and interior masonry partitions, we then considered three structures, of two, four and eight storeys. Each structure is analysed as a bare frame with two different infill configurations (fully filled and partially filled), as shown in Figures 3 to 5. The dimensions in plan, \((6.00 \times 6.00) \text{ m}^2\) and the story height is \(3.00 \text{ m}\), are the same for the three structures.

![Figure 3. Structure 1: Two storey](image)

![Figure 4. Structure 2: Four storey](image)

![Figure 5. Structure 3: Eight storey](image)

2.5. Pushover Analysis

Pushover analysis is a nonlinear static analysis that involves the application of gravity loads and a representative lateral load model. The frames were subjected to simultaneous side loading and gravity loads, the latter being in place during side loading. Lateral forces were applied monotonically in a step-by-step static nonlinear analysis. The applied lateral load model consists of a unit of acceleration multiplied by the mass at each stage level. In nonlinear static analysis, the capacity curve represents the relationship between base shear and roof displacement as well as characterizes the behaviour of the structure.

3. Results and Discussions

3.1. Results

The results of the nonlinear static analysis of the present study show that the hinge state indicates the level of performance of the structure identified by the figure of force-displacement curves of the hinges with colour codes as shown in Figure 2. At each step, the location of the plastic hinges is shown in Figures 6 to 14. The plastic hinges in the
bare frames are distributed over the height of the structures, while in the filled frames; the plastic hinges tend to be concentrated in the lower levels, in particular for solid infills. For the bare two-storey frame, the beams of all spans do not exceed the performance level (IO), for the ground floor, but for the columns, the performance level exceeds (CP), for the ground floor, as shown in Figure 6.

![Figure 6. Location of plastic hinges on two storey bare frames](image)

For the reinforced concrete framework, with two floors, totally filled, the beams of all the spans do not exceed the performance level (IO), for the ground floor and the first floor, but for the columns, the performance level does not exceed (CP), for the ground floor, as shown in Figure 7.

![Figure 7. Location of plastic hinges on two storey fully infilled frames](image)

For the reinforced concrete frame, two-storey, partially filled, the beams of all the spans do not exceed the performance level (IO), but for the columns, the performance level does not exceed (CP), for the ground floor, as shown in Figure 8.

![Figure 8. Location of plastic hinges on two storey partially infilled frames](image)

For the bare four-storey frame, the beams of all spans do not exceed the performance level (IO), for the ground floor and the first floor, but for the columns; the performance level does not exceed (CP), for the ground floor and the first floor, as shown in Figure 9.
Figure 9. Location of plastic hinges on four storey bare frames

For the reinforced concrete framework, with four floors, totally filled, the beams of all the spans do not exceed the performance level (IO), for the ground floor and the first floor, but, for the columns, the performance level does not exceed (CP), for the ground floor and the first floor, as shown in Figure 10.

Figure 10. Location of plastic hinges on four storey totally infilled frames

For the reinforced concrete framework, with four floors, partially filled, the beams of all the spans do not exceed the performance level (IO), for the ground floor, but for the columns, the level of performance does not exceed (CP), for the ground floor, as shown in Figure 11.

Figure 11. Location of plastic hinges on four storey partially infilled frames
For the bare eight-storey frame, the beams of all spans do not exceed the performance level (CP), for the ground floor and the seven floors, but for the columns, the performance level does not exceed (LS), for the for the ground floor and the three floors, as shown in Figure 12.

Figure 12. Location of plastic hinges on eight storey bare frames

For the reinforced concrete frame, with eight floors, totally filled, the beams of all the spans do not exceed the performance level (IO), for the ground floor and the seven floors, however, for the columns, the performance level does not exceed (LS), for the ground floor and the first floor, as shown in Figure 13.

Figure 13. Location of plastic hinges on eight storey totally infilled frames

For the reinforced concrete frame, with eight floors, partially filled, the beams of all the spans do not exceed the performance level (IO), for the ground floor and the seven floors, but for the columns, the performance level does not exceed (CP), for the ground floor and the seven floors, as shown in Figure 14.
3.2. Discussions

A Pushover Analysis, known as nonlinear static analysis, was carried out on three reinforced concrete frames of two, four and eight floors. Each structure is analysed as a bare frame with two different infill configurations (totally filled and partially filled). The capacity curves and the distribution of the plastic hinges in the different structures are the important results that can be obtained from this analysis, as shown in Figures 15 to 17. The capacity curves show that the infill walls allow the frames to support greater lateral loads. The patterns of the hinges are influenced by the presence of infills according to the dynamic characteristics of the structures.

Figure 14. Location of plastic hinges on eight storey partially infilled frames

Figure 15. Capacity curves of two storey building frames
For two-storey buildings, the fully filled frame results in an increase in stiffness and strength respectively by factors of: 4.0, 7.8 compared to the bare frame. These factors are at values equal to 3.72, 3.50 for four-storey buildings and 2.0, 3.0 for eight-storey buildings, respectively, relative to the bare frame. The addition of masonry infill walls increases both the rigidity and strength of the structures. The responses of the two- and four-storey structures are somewhat different from that of the eight-storey structure. The thrust curves of the fully filled reinforced concrete frame and the bare frame are identical, showing that after the filling has broken, the building's response is that of the bare frame. The behaviour of the partially filled reinforced concrete frame lies between the case of the fully filled reinforced concrete frame and the case of the bare frame with a sudden decrease in masonry infill walls have a significant effect on the seismic response of reinforced concrete framing. In four- and eight-storey reinforced concrete frame structures, a strong decrease in strength for the fully filled frame which can be attributed to brittle fracture of the masonry. The presence of masonry walls has a significant effect on the observed collapse mechanism.

4. Comparative Studies

In any scientific research, it is suggested to compare with other studies to be able to assess the authenticity of the work carried out. Thus, we could be offered a comparison of similar results made by researchers worldwide with our study finding carried out by the analysis nonlinear static (Pushover Analysis). Our study (named A) is based on three reinforced concrete structures of two, four and eight storey. That of Mociran & Cobirzan (2021) [3] (named B) is the study of a five-storey reinforced concrete framework. The third, (named C) of Tavakoli & Akbarpoor (2014) [28] deals with reinforced concrete buildings with five and ten storey and finally the last (named D) undertaken by Noui et al. (2019) [25], this study concerns three reinforced concrete frames of two, five and ten storey. These various reinforced concrete frames are designed in accordance with the regulations in use with two, three spans of 4.00 m to 6.00 m in length and the height of the floor is identical ranging from 3.00 m to 3.20 m.

The main results obtained from such studies, for a nonlinear static analysis (Pushover Analysis) by evaluating the seismic performance of buildings are:
• For two-storey buildings, study (A) concluded that the fully filled frame results in an increase in stiffness and strength respectively by factors of 4.0 and 7.8 compared to the bare frame.

• While, study (D) states that the existence of masonry infill panels in a frame increases the structural strength and rigidity compared to a bare frame, but at the same time the interaction must be taken into account. Similarly, the difference in the fundamental period between the bare frame (100% opening) and the fully filled frame (0% opening) is around 27%.

• For four-storey buildings, study (A) has shown that the fully filled frame results in an increase in stiffness and strength respectively by factors of 3.72 and 3.50 compared to the bare frame. Also, it should be noted that for these structures, there is a strong decrease in resistance for the completely filled frame which is attributed to the brittle fracture of the masonry.

• For five-storey buildings, study (B) concludes that the infills have been completely destroyed in the first three floors and on the fourth level only a partial damage noticed. While study (C) states that the failure of reinforced concrete elements is localized in the lower floors and limits the spread of local damage and it is evident that increasing the level of the floor improves the performance of the structure, in particular, in the lower floors. On the other hand, study (D) confirms that the difference in the fundamental period between the bare frame (100% opening) and the fully filled frame (0% opening) is about 31%. Similarly, the percentage difference in resistance capacity between the fully filled frame and the bare frame is around 84%.

• For eight-storey buildings, study (A) concludes that the fully filled frame results in an increase in stiffness and strength respectively by factors of 2.0 and 3.0 compared to the bare frame, as it is important to note that for these structures a strong decrease in strength for the fully filled frame attributed to brittle fracture of the masonry is observed. While, the response of the eight-storey structure is somewhat different from that of the two- and four-storey structures.

• Finally, for ten-storey buildings, study (D) approves that the difference in the fundamental period between the bare frame (100% opening) and the fully filled frame (0% opening) is near of percentage of 37%. Likewise, the percentage difference in resistance capacity between fully filled and bare frames is higher about 82%.

These various studies have shown that the addition of infill walls increases both the rigidity and the strength of the structure. The behaviour of the structural members is similar in all cases, with plastic hinges at the ends of the beams in the first three levels and at the base of the columns. The existence of a filler panel prevents the progression of the failure by limiting its development in a localized area. The evaluation of the capacity curves and the shear strength index (R) of the frames studied shows that the addition of infill panels increases the shear strength of the structure and improves the performance of structures in the upper floors, by preventing the propagation of the failure development and by locating the damage imposed on the lower floors. We have noticed, finally, an increase in the initial rigidity and the resistance capacity of the filled reinforced concrete frame compared to the bare frame, despite the brittle failure modes of the masonry walls.

5. Conclusion

A nonlinear static analysis (Pushover Analysis) was carried out on three reinforced concrete frames of low, medium and higher levels, the infill panels are modelled using a nonlinear stratified shell with the constitutive law of the masonry. The main results of which are the capacity curves and the distribution of the hinges in the different structures. These results indicate that the presence of the non-structural masonry infill can significantly alter the seismic response of reinforced concrete frames. The implemented infill walls increases the rigidity and resistance of the structures. The rigidity and strength of the fully filled frame is greatly improved despite the fact that masonry infills can show brittle failure. The response of the eight-storey structures is somewhat different from that of the two- and four-storey structures. The patterns of the hinges are influenced by the presence of infills according to the dynamic characteristics of the structures. The capacity curves show that the infill walls allow the frames to support greater lateral loads. The behaviour of the partially filled frames is close to that of the bare frames except for the eight-storey building case. The thrust curves of the bare frame and the fully filled frame are identical, showing that after the infill breaks, the building's response is that of a bare frame structural element. Finally, it could be that the contribution of infill panels should therefore be considered since they can have a positive or negative effect.

6. Declarations

6.1. Author Contributions

Conceptualization, A.Z. and A.K.; methodology, A.Z.; software, A.Z.; validation, A.Z., A.K. and Ab.Z.; formal analysis, A.Z. and A.K.; investigation, A.Z.; resources, A.Z. and A.K.; data curation, A.Z.; writing—original draft preparation, A.Z.; writing—review and editing, A.Z., A.K. and Ab.Z.; visualization, A.Z.; supervision, A.Z., A.K. and Ab.Z.; project administration, A.Z.; funding acquisition, A.Z. 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

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6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Trend Analysis of Meteorological Variables: Rainfall and Temperature

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Abstract

The Souss-Massa region in southwestern Morocco is characterized by a semi-arid climate with high variability in rainfall. Frequent droughts and flash flood events combined with overexploitation of water resources in recent decades have had a significant impact on the human security and the economy which is mainly based on agriculture, tourism, and fishery. For better management of extreme events and water resources under changing climatic conditions, a study was carried out to quantify the seasonal and annual variability and trends in rainfall and temperature over the past three decades with data from three stations. Climatological representative of the Souss-Massa region. The Mann-Kendall (MK) non-parametric test and the Sen’s slope are used to estimate the monotonic trend and magnitude of the trend of the variables, respectively. Statistical analysis of the rainfall series data set highlights that the occurrence of rainfall is unpredictable and irregular and the both the seasonal and annual rainfall trend appears negative (downward) for all the three climatological stations. The minimum temperature shows a remarkable increasing trend both on annual and seasonal scale while the maximum temperature registers a slight increasing trend. The study presents some new insights on rainfall and temperature trends that will have significant impacts on the surface and groundwater resources of the region under changing climatic conditions. The results can help to prioritize new strategies to mitigate the risk of droughts, of floods and to manage water resources to sustain the dependence of agriculture tourism and fishery sectors in the region.

Keywords: Rainfall; Temperature; Mann-Kendall Test; Sen’s Slope Estimator; Trend Analysis; Souss Basin.

1. Introduction

The impact of climate change on rainfall and air temperature has received much attention by the research community all around the world. Several studies have been carried out to show these changes in temperature and rainfall are becoming evident on a global scale [1]. Climate change has occurred on a global scale, but its impact often varies from region to region [2]. Hence, analyzing the change in meteorological variables represents an important task for climate change detection. As proved by numerous studies, climate change has a very strong impact on natural ecosystems, society and economy by affecting the hydrological cycle, which can lead to deficiency of water resources, and overabundance of floods and droughts frequency [3]. Due to global warming, there are strong signals of changes in rainfall are already occurring globally and locally [4].

Several studies of time series data have shown that the trend is either decreasing or increasing for both temperature and rainfall. We can define trend as the speed and direction in which individual data in a time series changes. In order
to understand a phenomenon, it is important to collect the data over a desirable period of time or space and process it in order to correct gaps and missing data, it is then interpreted to study the behavior of the phenomenon. The trend is analyzed by many parametric and non-parametric methods. Parametric methods, for example, the linear regression test, the graphing method, and the least squares method. Non parametric tests, are such as Spearman's Rho (SR) and Mann-Kendall Method and Sen’s slope estimator test which are the best known and most widely used among non-parametric tests to identify trends in climatic parameters.

Time series trend analysis involves analyzing the magnitude of the trend and its statistical significance. In various studies analyzing climate trends the following researchers; [5, 6] explained the reasons for using non parametric tests instead of parametric methods. Parametric methods are developed on the basis of assumptions, such as normality, stationarity and independence of time series, while these assumptions are the rarest to be satisfied in a climatic and hydrological dataset. In addition, parametric tests are very sensitive to the presence of outliers in the data series, which is not the case with non-parametric methods.

Morocco belongs to one of the six regions most affected by climate change, which is noticeable in extreme weather events, such as droughts and floods [7, 8]. In recent decades, researchers have observed a trend towards climatic aridity, which could lead to a decreasing of water resources. The water resource in the Souss basin has a very socioeconomic importance, since the economic activity is mainly based on agriculture and livestock activities. Another major climate risk that threatens the Souss basin is flashfloods; this phenomenon can cause loss of human lives and severe damages to infrastructures. This risk is becoming more and more frequent in the Souss basin, threatening the population, infrastructure and agricultural fields located on the banks of the Souss River [9].

At the end of February 2018 in the Souss basin, the rainfall had been evaluated at 150 mm, spread over 10 days, in November 2014 the rainfall had been evaluated at 150 mm spread over three days, in 2010 a flood place whose return period has been estimated at 114 years (Reports source ABHSM), [10], rare floods seem to become more and more frequent and cause more and more serious damage. Our study is focused on studying climate variables such as temperature and rainfall and forecasting their future trend and its impact on water resources. It is crucial to take these issues into consideration, by policy makers managers and planners when developing new mitigation strategies concerning water resource estimation and management of extreme events such as droughts and floods, for the Souss region.

In this study, the distribution of the time series is unknown, that is why we tried to apply the Mann-Kendall method to detect the trend of precipitation, minimum and maximum temperatures and the magnitude of the trends by the method of the Sen Slope estimator. Three stations were taken into account for the analysis. Before the trend analysis the data were subjected to missing data treatment in previous studies [11, 12].

2. Study Area

The Souss basin is located in the western zone of southern High Atlas of Morocco occupying a total area of 16200 km², and dominated by an arid to semi-arid climate. The dataset used in this study measured in Station ‘PT; Pont Taroudant’, ‘PA; Pont Aoulouz’ and ‘BA; Barrage. Abdelmoumen’ (See Figure 1).
3. Material and Methods

Data set used in the study; are Annual and seasonal rainfall data over a period of 40 years, ranging from 1981 to 2020. Annual and seasonal minimum and maximum temperatures over 36 years from 1981 to 2016. These data were collected from the Hydraulics Basin Agency of the region of Souss Massa ABHDSM. The Mann-Kendall and Sen’s slope estimator test was performed at a significance level of 5% on the average date of meteorological series from three stations. According to the change of temperature for an entire year, four seasons named winter (December to February), spring (March to May), summer (June to August) and fall (September to November) were used.

3.1. Mann–Kendall Test

The Mann Kendall test is a widely used statistical test for analyzing trends in climatological [13] and hydrological [14] data sets. There are two advantages to use this test. First, it is a nonparametric test and does not require the data to be normally distributed. Second, the test has low sensitivity to sudden breaks due to non-homogeneous time series [15]. All data reported as unobserved are included by assigning it, a common value smaller than the smallest measured value in the dataset [16].

According to this test, the null hypothesis $H_0$ assumes that there is no monotonic trend (the data are independent and randomly ordered) and this is tested against the alternative hypothesis $H_1$, which assumes that there is a monotonic upward or downward trend in climate time series data [17]. The trend can be assumed to be monotonic when (mathematically speaking, the trend is constantly increasing and never decreasing or never decreasing and never increasing).

Mann kendall test performs two kinds of statistics according to the number of data values, that is, S - statistic is used if the number of data values is less than 10 while Z - statistic (approximation / normal distribution ) for data values greater than or equal to 10. The Mann Kendall test calculation process considers the time series of n data points and Ti and Tj as two data subsets where $i = 1,2,3,…$, n-1 and $j = i + 1, i + 2, i + 3,…$, n. Data values are evaluated as an ordered time series. Each data value is compared against all subsequent data values.

Figure 2. Methodology flow chart of the study
If a data value of a later period is greater than a data value of a previous period, the $S$ statistic is incremented by 1. On the other hand, if the data value of a later period of time is less at a previously sampled data value, $S$ is decremented by 1. The net result of all these increments and decrements gives the final value of $S$ [18]. Thus the Mann-Kendall statistic $S$ is calculated as shown in Equation 1:

$$S = \sum_{i=1}^{n-1} \sum_{j=i+1}^{n} \text{sign}(T_j - T_i)$$

(1)

where, $T_j$ and $T_i$ are annual values in years $j$ and $i$, $j>i$ respectively, $n$ is the number of data points and $\text{sign}(T_j - T_i)$ is calculated using Equation 2:

$$\text{sign}(T_j - T_i) = \begin{cases} 1 & \text{if } T_j - T_i > 0 \\ 0 & \text{if } T_j - T_i = 0 \\ -1 & \text{if } T_j - T_i < 0 \end{cases}$$

(2)

If the number of data values is less than 10, the value of $|S|$ is compared directly to the theoretical distribution of $S$ derived by Mann and Kendall. The bilateral test is used. At some level of probability, $H_0$ is rejected in favor of $H_1$ if the absolute value of $S$ is equal to or exceeds a specified value $S_{\alpha}/2$, where $S_{\alpha}/2$ is the smallest $S$ which has the probability less than $\alpha/2$ (appear in case of no trend).

A very high positive value of $S$ is an indicator of an uptrend, and a very low negative value indicates a downtrend [19]. If the number of data values is equal to or greater than 10, the $S$ statistics behave approximately as normally distributed and the test is performed with a normal distribution with the mean and variation as shown below in Equations 3 and 4.

$$E(S) = 0$$

(3)

$$\text{Var}(S) = \frac{n(n-1)(2n+5)-\sum t_i(i)(i-1)(2i+5)}{18}$$

(4)

where $t_i$ is the number of links in range $i$ (zero difference between compared values). The sum term in the numerator is used only if the data series contains linked values. The standard test statistic $Z$ is calculated using Equation 5.

$$Z = \begin{cases} \frac{S-1}{\sqrt{\text{Var}(S)}} & \text{if } S > 0 \\ 0 & \text{if } S = 0 \\ \frac{S+1}{\sqrt{\text{Var}(S)}} & \text{if } S < 0 \end{cases}$$

(5)

The test is used for four $\alpha$ significance levels: 0.1, 0.05, 0.01 and 0.001. The 0.05 significance level means that the existence of a monotonic trend is very likely. Respectively, the significance level 0.1 means that there is a 10% probability that we will make an error when we reject $H_0$.

It is necessary to calculate the probability associated with $S$ and the sample size, $n$, to statistically quantify the significance of the trend. The procedure for calculating this probability will be described below. The probability value $P$ of the MKS statistic of the sample data can be estimated using the normal cumulative distribution function as

$$P = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{Z} e^{-t^2/2} \, dt$$

(6)

Based on a significance level of 5%, if the $p$-value is $\leq \alpha = 0.05$, then the $H_1$ hypothesis is accepted which means the presence of a trend in the data and if the $p$-value is $\geq \alpha = 0.05$ then $H_0$ will be accepted that denotes the absence of trend in the data.

### 3.2. Sen’s Slope Estimator

The non-parametric Sen’s slope estimator was developed by Sen in 1968, it is used to predict the magnitude (true slope) of hydrologic and meteorological time series data. It has been widely used to calculate the magnitude of trends in long-term temporal data [20, 21]. In this study, the Sen’s slope is applied to calculate the magnitude of the trend of the temporal data. It is considered to be better at detecting the linear relationship because it is not affected by outliers in the data. It uses a linear model for trend analysis. The slope ($T_i$) of all data pairs is calculated using Equation 7 [22].

$$T_i = \frac{x_j - x_k}{j-k}$$

(7)

Pour $i=1,2,3,\ldots,n$ where, $x_j$ and $x_k$ are data values at time $j$ and $k$ ($j>k$) separately.

The median of these $n$ values of $T_i$ is represented by the estimated slope of Sen (true slope) which is calculated using Equation 8.

$$Q_i = \begin{cases} \frac{T_{n+1}}{2} & \text{for } n \text{ odd} \\ \frac{1}{2} \left( \frac{T_n + T_{n+1}}{2} \right) & \text{for } n \text{ even} \end{cases}$$

(8)
Positive Qi values indicate an increasing trend, while negative Qi values indicate a decreasing trend in weather data. The unit of the slope of Sen Qi is the magnitude of the slope per year. The software used to perform the Mann-Kendall statistical test and Sen’s slope estimator is the Addinsoft 2020.1.3 XLSTAT. The null hypothesis is tested at a 95% confidence level for both the temperature and precipitation data.

We also obtain Kendall’s Tau, when running the Mann-Kendall test, which is a measure of correlation and then it measures the strength of the relationship between the two variables. Kendall's tau, will take values between ± 1 and 1, with a positive correlation indicating that the ranks of the two variables increase together while a negative correlation indicates that when the rank of one variable increases, the other decreases [1].

\[
\text{Kendall’s tau} = \frac{C - D}{C + D}
\]

where; C: Concordant pairs the number of observed ranks below a particular rank which are larger than that particular rank, and D: Discordant pairs the number of observed ranks below a particular rank which are smaller in value than that particular rank.

4. Results and Discussions

The analysis of trends in climatological variables of the Souss basin was carried out in the first phase with rainfall data over 40 years from 1981 to 2020 and in the second phase, on minimum and maximum temperature data over a period of 36 years since 1981 to 2016. For results interpretation we have made two assumptions:

- \( H_0 \): There is no monotonic trend in the series;
- \( H_1 \): There is a trend in the series.

So, if the calculated p-value is greater than the level of significance alpha = 0.05, we need to accept the null hypothesis \( H_0 \). First of all, we defined the 5% threshold with our null hypothesis (there is no trend), so we judged the trend according to this threshold. However, there may be a trend in the data behind the selected threshold. For example, we take the rainfall series of the PT station during the autumn season from Table 1, at the significance level of 5%, we got the p-value = 0.571 so , based on the null hypothesis, we have to say that there is no trend at a significance level of 5%. However, the statistic of the Mann kendall test \( S = -135 \), reveals that there is a negative trend which is significant at a significant level of 10%. Therefore, to determine the trend the Sen slope estimator \( Q \) should be applied to determine the magnitude of the trend.

4.1. Phase 1 Applying Tests on Rainfall Data

The Mann kendall test and Sen’s slope estimator were applied seasonal and annual rainfall datasets.

**Seasonal Rainfall Data:**

<table>
<thead>
<tr>
<th>Stations</th>
<th>Saison/Test</th>
<th>Kendall’s Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>Q Sen’s slope</th>
<th>alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>Winter</td>
<td>-0.227</td>
<td>7365.6</td>
<td>0.039</td>
<td>-1.356</td>
<td>0.05</td>
<td>Reject ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.064</td>
<td>7366.6</td>
<td>0.571</td>
<td>0.329</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>-0.173</td>
<td>7365.6</td>
<td>0.116</td>
<td>-0.673</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.118</td>
<td>5533.3</td>
<td>0.333</td>
<td>0.000</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td>PA</td>
<td>Winter</td>
<td>-0.277</td>
<td>7366.6</td>
<td>0.012</td>
<td>-2.121</td>
<td>0.05</td>
<td>Reject ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.023</td>
<td>7366.6</td>
<td>0.844</td>
<td>0.228</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>-0.062</td>
<td>7366.6</td>
<td>0.586</td>
<td>-0.397</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>-0.071</td>
<td>7355</td>
<td>0.521</td>
<td>-0.039</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td>BA</td>
<td>Winter</td>
<td>-0.138</td>
<td>7366.6</td>
<td>0.214</td>
<td>-1.089</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>-0.037</td>
<td>7365.6</td>
<td>0.735</td>
<td>-0.326</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>-0.201</td>
<td>7365.6</td>
<td>0.067</td>
<td>-1.103</td>
<td>0.05</td>
<td>Reject ( H_0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.014</td>
<td>6871.3</td>
<td>0.904</td>
<td>0.000</td>
<td>0.05</td>
<td>Accept ( H_0 )</td>
<td></td>
</tr>
</tbody>
</table>
Annual Rainfall Data:

Table 2. Mann Kendall’s test and Sen’s slope estimator on annual rainfall data

<table>
<thead>
<tr>
<th>Stations</th>
<th>Annual rainfall (mm)</th>
<th>Kendall’s Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>Q Sen’s slope</th>
<th>alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>189.6</td>
<td>-0.136</td>
<td>-106</td>
<td>7366.6</td>
<td>0.223</td>
<td>-1.242</td>
<td>0.05</td>
<td>Accept H₀</td>
</tr>
<tr>
<td>PA</td>
<td>305.1</td>
<td>-0.121</td>
<td>-94</td>
<td>7366.6</td>
<td>0.281</td>
<td>-1.823</td>
<td>0.05</td>
<td>Accept H₀</td>
</tr>
<tr>
<td>BA</td>
<td>337.1</td>
<td>-0.146</td>
<td>-114</td>
<td>7366.6</td>
<td>0.189</td>
<td>-2.952</td>
<td>0.05</td>
<td>Accept H₀</td>
</tr>
</tbody>
</table>

Figure 3. Presents the Annual rainfall trend and presentation of Ten-year polynomial regression forecast

The Sen’s slope estimator revealed a slight downward trend in seasonal and annual rainfall the results are shown in Tables 1 and 2 for each station and Figures 3a, 3b and 3c. To evaluate the temporal variable of the annual rainfall of the stations represented, we carried out a trend study using a ten-year polynomial regression forecast, the 2nd degree polynomial trend curve is the one which better represents the tendency of the annual rainfall to be measured in the
stations, and moreover it has the biggest coefficient of determination $R^2$, even if it is still small. Results are shown in Figures 3a 3b and 3c. The Figures show a sawtooth rainfall evolution over 40 years with a slight downward trend in the slope of Sen, which is explained by the irregularity of the rainfall regime in this area. The area is mostly attacked by long periods of droughts interspersed with flash flood events. The years in which there is an increase in rainfall correspond to flash flood events according to reports from the Souss Massa hydraulic basin agency.

For all the three station PT, PA and BA; year 2009 recorded high rainfall values during 40 years; 561, 826, and 1091 mm respectively. The entire region was devastated by floods according to the Souss Massa hydrological agency reports. Rainfall values never reached that level and tended to decrease according to Q and Mann kendall S values Table 2. The polynomial curve predicts a decrease over the next 10 years. According to the report of Morocco’s 3rd national communication to the United Nations Framework Convention on Climate Change, predict a downward trend in annual cumulative rainfall that varies between 10 and 20% to reach 30% over regions of the arid and saharian climate by 2100 [22]. In Previous studies the distribution of annual precipitation mean during 1980-2010 shows that it rains remarkably more from the North to the South and from the West to the East in the Souss Massa Region. The dry months are defined as months with precipitation below 20 mm and the wet months are those that exceed of 20 mm [23].

### 4.2. Phase 2 Applying Tests on Temperature Data

#### 4.2.1. Minimum Temperature Data

Table 3 shows the results of the Mann kendall test and the Sen’s slope estimator applied on the seasonal minimum temperature data for each station. P-values are less than $\alpha = 0.05$ and large positive values of S, consequently confirm a strong increasing tendency of the minimum temperatures. The seasonal minimum temperature has been rising for the past 36 years. Mann kendall test and the Sen’s slope estimator were applied on the minimum temperature values of each year in order to know the annual trend of the minimum temperatures.

Results are shown in Table 4; according to p-values, we have accepted hypothesis $H_0$, saying that there is an upward trend, according to the positive values of S we say that the trend is strong. Figs. 4a 4b and 4c show the variation in the minimum temperature during the 36 years, in each of the stations PT, PA and BA; the Sen curve shows a notable increase; the polynomial curve shows the forecast by regression 10 years ahead, we notice that the increase will always be maintained. In the station PT, Figure 4(a), the minimum temperature has been rising over the past 36 years, according to the Sen’s slope trendline and S value presented in Table 4. According to the forecast polynomial curve this temperature will decrease by 1°C in 2026. The hottest year was in 2002 with a minimum temperature of 6°C. The coldest year was 1981 the minimum temperature was -3°C. Since 2010 the minimum temperature oscillated between 3 and 3.2°C.

At the PA station Figure 4(b) the minimum temperature has never recorded a value less than 0 °C during the past 36 years, it has stabilized around 5 °C since 2008. The Sen Curve shows a noticeable increase which is confirmed by S value Table 4. According to the polynomial curve of ten year forecasting the minimum temperature will continue to increase. Minimum temperatures in the BA station recorded large values; the highest minimum temperature was 10°C recorded in 2010 during the 36 years. The minimum temperature was still increasing according to Sen’s slope Figure 4(c) and S-values Table 6. According to the polynomial curve representing the 10 years forecasting the minimum temperature will always be rising.

<table>
<thead>
<tr>
<th>Stations</th>
<th>Saison/Test</th>
<th>Kendall’s Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>Q Sen’s slope</th>
<th>alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>Winter</td>
<td>0.439</td>
<td>276</td>
<td>5388</td>
<td>0.000</td>
<td>0.096</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.303</td>
<td>190</td>
<td>5385.3</td>
<td>0.010</td>
<td>0.050</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.277</td>
<td>174</td>
<td>5384.3</td>
<td>0.018</td>
<td>0.050</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.358</td>
<td>225</td>
<td>5387</td>
<td>0.002</td>
<td>0.055</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td>PA</td>
<td>Winter</td>
<td>0.486</td>
<td>306</td>
<td>5386</td>
<td>&lt;0.0001</td>
<td>0.042</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.177</td>
<td>111</td>
<td>5387</td>
<td>0.130</td>
<td>0.025</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.313</td>
<td>196</td>
<td>5381.3</td>
<td>0.008</td>
<td>0.035</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.287</td>
<td>180</td>
<td>5385.3</td>
<td>0.014</td>
<td>0.025</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td>BA</td>
<td>Winter</td>
<td>0.489</td>
<td>306</td>
<td>5371.3</td>
<td>&lt;0.0001</td>
<td>0.163</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.125</td>
<td>78</td>
<td>5379.3</td>
<td>0.288</td>
<td>0.063</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.540</td>
<td>337</td>
<td>5375.6</td>
<td>&lt;0.0001</td>
<td>0.109</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.511</td>
<td>319</td>
<td>5372.3</td>
<td>&lt;0.0001</td>
<td>0.171</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
</tbody>
</table>
Table 4. Mann Kendall’s test and Sen’s slope estimator on annual minimum temperature data

<table>
<thead>
<tr>
<th>Stations</th>
<th>Annual Minimum Temperature (°C)</th>
<th>Kendall’s Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>Q Sen’s slope</th>
<th>alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>1.717</td>
<td>0.464</td>
<td>289</td>
<td>5371</td>
<td>&lt;0.0001</td>
<td>0.089</td>
<td>0.05</td>
<td>Reject H$_0$</td>
</tr>
<tr>
<td>PA</td>
<td>3.657</td>
<td>0.530</td>
<td>331</td>
<td>5378</td>
<td>&lt;0.0001</td>
<td>0.089</td>
<td>0.05</td>
<td>Reject H$_0$</td>
</tr>
<tr>
<td>BA</td>
<td>4.378</td>
<td>0.372</td>
<td>232</td>
<td>5375</td>
<td>0.002</td>
<td>0.176</td>
<td>0.05</td>
<td>Reject H$_0$</td>
</tr>
</tbody>
</table>

Figure 4. Presentations of the annual minimum temperature trend and Ten-year polynomial regression forecast in three stations

4.2.2. Maximum Temperature Data

For the maximum temperature we note that for the station PT the decreasing tendency is slightly noticed in autumn and winter seasons and tend to increase during the spring and the summer which is explained by the typical Mediterranean climate of the region, with mild wet winters and hot dry summers. For the PA and BA stations,
respectively, there is a decrease during the winter and an increase during the spring. During all seasons there is a significant increase at the BA station, Table 5.

Table 6, represents the results of the tests on the maximum temperatures of each year, there is an explicit downward trend in the PT station and slight downward trends in the other stations. Figures 5a, 5b and 5c show the Sen’s slope trendline which is almost stable, a very slight evolution, but when we apply a forecast according to the polynomial regression model, we note a slight increase over the next 10 years. In station PT Figure 5(a) the lowest maximum temperature was in 2009 the recorded value was 34 °C, the highest value was 48 °C in 1986. The curve of Sen’s slope is almost stable we note a very light decrease according to Q and S values Table 6. Giving the polynomial curve the temperature will be maintained from 44 and 45 °C over the next 10 years.

For the station PA Figure 5(b) the lowest temperature was 41 °C in 1997. The highest value was 48 °C recorded in 1986. The curve of Sen tends almost towards a slight decrease according to Q and S values Table 6. Giving the polynomial curve the temperature will tend towards a remarkable increase in the next 10 years. For the station BA Figure 5(c) the lowest temperature was 25 °C in 2006 and 2005. The highest value was 51 °C in 2009. The curve of Sen is almost steady according to Q values table 6. Following the polynomial curve, the temperature will tend to increase in the next 10 years.

For annual average temperatures, an increasing trend of 0.5 to 1 °C is projected for 2020 and of 1 to 1.5 °C for 2050 and 2080, across the country. In general, an increase in minimum temperature will rise the capacity of the atmosphere to retain water through the phenomenon of evaporation and transpiration which could increase rainfall. In fact, in one year, most of the flows in the Souss valley appear in the form of violent and brief floods, resulting from heavy supply rains concentrating on a few days or a few months. Vulnerability to flash floods could be further intensified by alternating periods of drought and episodes of extreme rainfall, which could also lead to soil saturation as well as problems with runoff and soil erosion. An aggravation of the geotechnical risks of foundations (collapses linked to very rainy episodes).

According to the above findings, it is extremely important to discuss the ecological, economic and social impacts that could result from continued decreasing trends in rainfall. In this region of an arid to semi-arid climate, drought events will be more and more serious; this could have severe impacts on surface water resources sustainability and groundwater recharge. The deficit in groundwater is very pronounced; the renewable potential of the Souss aquifer will decrease by 43% according to climate models; The marine intrusion has taken the lead in the coastal strip where the salinity continues to increase. Many sources and khettaras have lost their flow, or even dried up; Reservoirs of dams attest to a considerable loss by evaporation during these last years. And also, siltation is a phenomenon that could result from continued decreasing trends in rainfall. In this region of an arid to semi-arid climate, drought waves that could be difficult for the elderly and other vulnerable populations by increasing probabilities of cardiovascular and respiratory risks. Intensifies infectious and chronic diseases, especially waterborne diseases (typhoid, viral hepatitis).

### Table 5. Mann Kendall’s test and Sen’s slope estimator on seasonal maximum temperature data

<table>
<thead>
<tr>
<th>Stations</th>
<th>Saison/Test</th>
<th>Kendall’s Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>Q Sen’s slope</th>
<th>alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>Winter</td>
<td>-0.186</td>
<td>117</td>
<td>5387</td>
<td>0.114</td>
<td>-0.029</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>-0.045</td>
<td>-28</td>
<td>5385</td>
<td>0.713</td>
<td>-0.007</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.175</td>
<td>110</td>
<td>5386</td>
<td>0.137</td>
<td>0.039</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.118</td>
<td>74</td>
<td>5386</td>
<td>0.320</td>
<td>0.012</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Winter</td>
<td>-0.237</td>
<td>-149</td>
<td>5387</td>
<td>0.044</td>
<td>-0.031</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.048</td>
<td>30</td>
<td>5385</td>
<td>0.693</td>
<td>0.005</td>
<td>0.05</td>
<td>Accept H0</td>
</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.457</td>
<td>287</td>
<td>5387</td>
<td>&lt;0.0001</td>
<td>0.060</td>
<td>0.05</td>
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</tr>
<tr>
<td></td>
<td>Summer</td>
<td>-0.017</td>
<td>-11</td>
<td>5389</td>
<td>0.892</td>
<td>-0.003</td>
<td>0.05</td>
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</tr>
<tr>
<td></td>
<td>Winter</td>
<td>0.407</td>
<td>256</td>
<td>5388</td>
<td>0.001</td>
<td>0.282</td>
<td>0.05</td>
<td>Reject H0</td>
</tr>
<tr>
<td></td>
<td>Autumn</td>
<td>0.429</td>
<td>270</td>
<td>5388</td>
<td>0.000</td>
<td>0.220</td>
<td>0.05</td>
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</tr>
<tr>
<td></td>
<td>Spring</td>
<td>0.643</td>
<td>404</td>
<td>5386</td>
<td>&lt;0.0001</td>
<td>0.252</td>
<td>0.05</td>
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</tr>
<tr>
<td></td>
<td>Summer</td>
<td>0.446</td>
<td>280</td>
<td>5385.3</td>
<td>0.000</td>
<td>0.167</td>
<td>0.05</td>
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</tr>
</tbody>
</table>
Table 6. Mann Kendall’s test and Sen’s slope estimator on annual maximum temperature data

<table>
<thead>
<tr>
<th>Stations</th>
<th>Annual Maximum Temperature (°C)</th>
<th>Kendall's Tau</th>
<th>S</th>
<th>Var(S)</th>
<th>p-value</th>
<th>QSen’s slope</th>
<th>Alpha</th>
<th>Test Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>45.6</td>
<td>-0.231</td>
<td>-144</td>
<td>5372.6</td>
<td>0.049</td>
<td>-0.045</td>
<td>0.05</td>
<td>Reject H₀</td>
</tr>
<tr>
<td>PA</td>
<td>43.2</td>
<td>-0.120</td>
<td>-75</td>
<td>5382.3</td>
<td>0.307</td>
<td>-0.011</td>
<td>0.05</td>
<td>Accept H₀</td>
</tr>
<tr>
<td>BA</td>
<td>42.2</td>
<td>-0.008</td>
<td>-5</td>
<td>5367</td>
<td>0.946</td>
<td>0.000</td>
<td>0.05</td>
<td>Accept H₀</td>
</tr>
</tbody>
</table>

![Trend of Tmax](image)

Figure 5. Presentations of the annual maximum temperature trend and Ten-year polynomial regression forecast in three stations

5. Conclusion

Based on the results of our study, we accept that the Mann-Kendall test and the Sen Slope estimator provide remarkable results on the presentation of the evolution of climate variables in the Souss basin. We have noticed an irregularity of rainfall and long periods of drought; we have noticed as well an obvious increasing trend of minimum temperature and a decreasing trend of rainfall in all the three stations. This Trend could be the main reason that
engenders the succession of extreme events such as drought and flash floods in the Souss region. In fact, an increasing temperature trend makes the atmosphere warmer; warm air can contain more water vapor than cold air. For every 1 °C increase in temperature, the air can hold about 7% more humidity, which condenses to fall as rain; the more the temperature rises, the more it causes heavy rains in a very short time interval. These results are confirmed by numerous studies and national and international reports on climate change, adopting that Morocco belongs to one of the six regions most affected by the effects of climate change,

The arid climate of the Souss basin favors the appearance of violent floods, due to; the occurrence of heavy rains in a short time, the absence of plant cover and the type of soil that favors runoff. Since droughts and flash floods are inevitable, it is advisable to observe and study the changes in temperature and rainfall in order to avoid plausible devastating consequences. Along with anticipating and searching for new metrics for adapting and attenuating these problematic phenomena, which have become more and more frequent. At the end we can recommend to determine the overflow zones for floods of different return periods, by mapping as an essential element to raise awareness, secure human lives, avoid damages and support decisions and actions.

6. Declarations

6.1. Author Contributions

Conceptualization, J.E. and A.L.; methodology, J.E.; validation, J.E. and A.L.; resources, H.S and I.J.; data curation, M.B.; writing—original draft preparation, J.E.; supervision, A.L.; 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

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6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Behavior of RC Wide Beams under Eccentric Loading

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Abstract

Wide beams are one of the widely used structural elements in RC buildings due to the many special features that characterize them. The main objective of this research is to investigate the behavior of wide shallow beams under the effect of eccentric loading acting along their cross sections. To achieve that, an experimental program that consisted of seven wide beams was conducted. All beams were loaded using two concentrated loads at their middle third where the main parameters considered were: the magnitude of the load eccentricity, the longitudinal spacing between shear reinforcement, and the arrangement of the longitudinal reinforcement. Following that, a finite element analysis was performed where the analytical model used was first verified using the data from the experimental program. The results from both the experimental and analytical programs were in good agreement. Then, the finite element analysis was extended through a parametric study where other variables were studied such as the compressive strength of concrete, the transverse spacing between stirrups and the longitudinal reinforcement ratio. The results showed that the value of the load eccentricity, spacing between shear reinforcement, the arrangement of the main reinforcement along the beam cross section, and the compressive strength of concrete significantly affected the torsional resistance of shallow wide beams. Conclusions and recommendations are presented which can be useful for future researchers.

Keywords: Shallow Wide Beam; Torsional Moment; Cracks Pattern; Eccentric Load; Ultimate Load; Finite Element.

1. Introduction

Using wide shallow beams presents a good solution for many architectural obstructions as it provides a better height clearance and more simplicity for internal partitioning, this is in addition to removing the potential obstacles in the way of electromechanical ductworks. Also, wide beams save construction time due to the simplicity of formwork and reinforcement detailing [1]. Generally, shear stresses are produced in any reinforced concrete cross section through shear forces or/torsion moments. Shear failure in concrete structures is catastrophic and happens suddenly, so it is a main concern for the design engineers. Once the tensile cracks occur in a beam, the shear capacity is reduced, so the designers control this issue by using conventional stirrups. On the other hand, the torsion stresses are always related to shear stresses which can be introduced to concrete structure by eccentric loading or eccentric support. It can clearly be seen in curved beams, spandrel beams and irregularly shaped sections. Practical situations where torsion in beams occurs are quite numerous, for example, when partitions or walls do not coincide with the centroidal longitudinal axis of the beam. This can occur in narrow beams but can be even more pronounced in wide beams due to their large width. Additionally, floor slabs cast monolithically with beams can cause torsional moments when they deflect under load. Torsion stresses lead to spiral or oblique cracks that extend all over the element which also can be improved or overcome by using well distributed vertical stirrups and main reinforcement steel bars that decrease the deflection and hair cracks, increase the ultimate load of failure, tensile strength, and ductility [2].

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Recently, most of the experimental and analytical investigators focused on studying the wide shallow beams in terms of their shear behavior. One of the most widely investigated topic is the effect of shear reinforcement on the shear strength of the wide beams. Kamal [3] studied the effect of shear reinforcement through three different parameters which are the volumetric ratio of vertical stirrups, concrete compressive strength, and beam width. He concluded that the vertical stirrups contribute greatly to the shear strength of wide beams. Said & Elrakib [4] tested nine wide beams with varying spacing and diameter of vertical stirrups as well as having different yield stresses. They showed that using shear reinforcement considerably enhances the shear capacity and ductility of wide beams, and that using high grade steel is more effective than mild steel. Mohammadian-Yasouj et al. [5] conducted an experimental program on six wide beams to study the use of different types of shear reinforcement. Vertical stirrups, independent bent bars, independent mid-depth horizontal bars, and a combination between bent up bars and vertical stirrups were used in this study. Ehab [6] carried out an experimental program to study the effect of shear reinforcement on the shear behavior and modes of failure of wide shallow beams. Four different parameters including shear reinforcement ratio, shear span to depth ratio, spacing between vertical stirrups, and number of vertical branches were investigated. The results showed that the vertical stirrups have a great contribution to the shear strength, ductility, and width of shear cracks. Transverse spacing between stirrups was proven to have a considerable effect on the shear strength of wide beams as shown by Lubell et al. [7]. While Shuraim [8] conducted a vast study on 16 two span continuous wide beams with various arrangement of stirrups in the transverse direction. He concluded that using four branch stirrups was more effective than two legged ones for the same amount of stirrups. Elansary et al. [9] suggested an innovative system of shear reinforcement for wide beams using confined spiral stirrups. They studied variable parameters of the stirrups such as spacing and ratio of stirrups. They also studied the effect of changing the longitudinal reinforcing bars on the behavior of the beams.

Concerns regarding design code formulation for shear in wide beams have been extensively studied. Angelakos et al. [10] investigated shear provisions provided by the ACI code [11] and showed that they can be quite un-conservative in the case of large beams or thick slabs. Sherwood et al. [12] conducted an experimental program to study the validity of the ACI-05 [13] recommendations. They stated that “the width of a member does not significantly affect the shear stress at failure and that the ACI 318-05 [13] provisions requiring different shear capacities for slabs, wide beams, and narrow beams are not appropriate.” Kim et al. [14] proposed an equation to calculate the shear strength of slabs depending on the spacing and support conditions. Collins et al. [15] studied the shear behavior of thick slabs without shear reinforcement. They found that the ACI code [16] provisions are highly underestimated in this case and that using minimum shear reinforcement improves the behavior of such thick slabs.

Torsion of reinforced concrete beams has been the topic of study for many years [17, 18]. Fang et al. [19] studied the size effect with regards to torsion in beams with and without stirrups. The properties of concrete as well as reinforcement generally affect the capacity of beams in torsion. Fang et al. [19] studied beams subjected to pure torsion using both normal and high strength concrete. They concluded that using high strength concrete improved the capacity and the cracked stiffness of the beams in torsion. On the other hand, Kim et al. [20] studied the effect of using stirrups with high strength reinforcement to resist torsional stresses in beams. They concluded that using high strength reinforcement in this case leads to lower capacity than the design values. Also, Lee et al. [21] studied the maximum torsion reinforcement with comparison between the different design codes regarding their accuracy. Different beam shapes were also studied, for example, Jeng et al. [22] studied large size hollow sectioned beams. Ju & Lee [23] studied the combined effect of torsion and bending. They tried to simulate the behavior of the beams under these stresses through an analytical model and comparison with previous experimental data.

2. Significance and Layout of the Research Program

Based on the above, considerable research was performed on either wide beams or the torsion behavior of narrow beams. However, scarce work can be found on the study of wide beams subjected to torsional effects. The main objective of this research is to study the effect of applying eccentric loading on shallow wide beams thus exerting torsional moments that lead to shear stresses and how the behavior is affected by changing different parameters through experimental as well as analytical analysis. The parameters under study were decided as those mostly affecting the wide beams or the torsion behavior of beams as reported in the literature; mainly the eccentricity of the load, the vertical stirrups spacing in the longitudinal direction and in the transverse direction, the arrangement of main longitudinal reinforcement, the compressive strength of concrete, and the ratio of the main longitudinal reinforcement. Torsion does not usually occur without any other action and thus the beams were all subjected to the combined action of torsion, shear, and bending through a two-point loading setup.

Figure 1 shows the layout of the research program. It is mainly divided into two parts namely the experimental program and the analytical one. The experimental program consists of testing seven wide shallow beam specimens under two-point loading and the numerical analysis was conducted through the finite element program ANSYS V.19 [24]. In the finite element analysis, first the seven specimens tested before were modeled for verification of the soundness of the modeling procedure used and then a parametric study was conducted to further understand the behavior of the wide beams in under eccentric loading.
3. Experimental Program

3.1. Specimens

The experimental program consisted of seven wide shallow beam specimens. A summary of the details of all beams is shown in Table 1. The loading setup was designed to introduce torsional moments on the specimens through the application of two vertical concentrated loads with varying eccentricities (e) of zero, 100 mm and 200 mm. These values correspond to e/B = zero, 0.167 and 0.333 where B is the width of the beam. The two-point loads were applied at the two middle thirds of the longitudinal span. All beams were casted and tested at the Concrete Research Laboratory, Faculty of Engineering, Cairo University. The seven specimens had the same cross section of 600 × 200 mm and the same span of 1700 mm with a clear span of 1500 mm. The clear concrete cover was 20 mm giving a beam depth d=180 mm. The ratio of the main bottom longitudinal reinforcement was taken as 0.78% which satisfies the minimum and maximum ratios set by ECP 203-2018 [25] and ACI 318-19 [26]. Figure 2a shows the details of all beams in plan where “Front” herein refers to the longitudinal side of the beam closest to the point load and back refers to the other side. Figure 2b shows the typical longitudinal cross section. Different arrangements were used for the bottom reinforcement; uniform distribution with two different diameters and different number of bars namely 4 and 7 and then banded type where part of the reinforcing bars was concentrated at the same side as the loading as shown in Figure 2c, 2d and 2e. The latter arrangement was chosen based on reports from previous research that banded reinforcement arrangement showed improvement in the capacity of the wide beams [5]. The top reinforcement was 4 bars with the diameter 10 mm. Stirrups with four branches of diameter 6 mm were used. Design codes [25, 26] limit the spacing between stirrups in the longitudinal direction to d/2. Based on this, the spacing in the longitudinal direction was taken as 50, 75, 100 mm, thus covering a range less than and close to the maximum spacing stipulation.
Table 1. Summary of the details of the beam specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dimensions</th>
<th>e (mm)</th>
<th>e/B</th>
<th>Bottom Reinforcement</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Area (mm²)</td>
<td>D (mm)</td>
</tr>
<tr>
<td>B1</td>
<td>600 x 200 x 1700 mm</td>
<td>0.0</td>
<td>0</td>
<td>804.24</td>
<td>16</td>
</tr>
<tr>
<td>Group (1)</td>
<td>B2</td>
<td>200</td>
<td>0.333</td>
<td>804.24</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>100</td>
<td>0.167</td>
<td>804.24</td>
<td>16</td>
</tr>
<tr>
<td>Group (2)</td>
<td>B4</td>
<td>200</td>
<td>0.333</td>
<td>804.24</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>B5</td>
<td>200</td>
<td>0.333</td>
<td>804.24</td>
<td>16</td>
</tr>
<tr>
<td>Group (3)</td>
<td>B6</td>
<td>200</td>
<td>0.333</td>
<td>791.68</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>B7</td>
<td>200</td>
<td>0.333</td>
<td>791.68</td>
<td>12</td>
</tr>
</tbody>
</table>

e: Load eccentricity along the cross section of the beams, B: width of the beam;
D: Diameter of reinforcement steel bars, s: Spacing between stirrups along the span of the beams.

Figure 2. Specimen details for Beams B1 to B7 (All dimensions are in mm)

Figure 3. Loading test setup for all beams
The concrete mix was designed to give a characteristic compressive strength of 30 N/mm². Compressive strength tests were conducted on standard cubes and gave an average strength of 30.6 N/mm². The loading was performed by means of a steel frame composed of groups of steel girders tied together by high strength steel bolts and nuts as shown in Figure 3. The load cell used had a capacity of 1000 kN. Three LVDTs were used to measure the deflection of the specimens. One LVDT was placed at the mid span of the specimens and the other two one under each load. Electrical strain gauges were used to measure the strain in the longitudinal reinforcement where they were placed at the maximum flexure point at mid-span. Strain gauges were also used in the vertical leg of the stirrups at d/2 from the face of support as shown in Figure 4. The position of strain gauges in this case were chosen at the location where the shear cracks are most expected to occur.

The beams were divided into three groups to study three parameters as follows:

- **Group (1):** This group consists of three beams – B1, B2, B3 – designed to study the effect of the load eccentricity along the cross section of the beams (e) varying from zero for B1, 200 mm for B2 and 100 mm for B3.
- **Group (2):** This group consists of two beams – B4, B5 in addition to beam B2 – where the spacing between the stirrups in the longitudinal direction (s) is studied with the beams subjected to a fixed eccentricity (e/B) of 0.333.
- **Group (3):** This group consists of two beams – B6, B7 in addition to beam B2 – to study the effect of the main longitudinal reinforcement arrangement as shown in Figure 2c through 2e.

### 3.2. Experimental Test Results

Table 2 shows the experimental output for all the beams in terms of the ultimate load, the first crack load, and the corresponding deflections as well as the calculated values of stiffness, ductility and toughness of each specimen and the modes of failure.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>P&lt;sub&gt;c&lt;/sub&gt; (kN)</th>
<th>P&lt;sub&gt;u&lt;/sub&gt; (kN)</th>
<th>Δ&lt;sub&gt;cr&lt;/sub&gt; (mm)</th>
<th>Δ&lt;sub&gt;uf&lt;/sub&gt; (mm)</th>
<th>ε&lt;sub&gt;cr&lt;/sub&gt; (μ)</th>
<th>ε&lt;sub&gt;u&lt;/sub&gt; (μ)</th>
<th>Stiffness (kN/mm)</th>
<th>Ductility factor</th>
<th>Toughness (kN.mm)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>80</td>
<td>320</td>
<td>1.1</td>
<td>14.5</td>
<td>4828</td>
<td>142</td>
<td>29.4</td>
<td>2.17</td>
<td>3426.8</td>
<td>Flexure</td>
</tr>
<tr>
<td>B2</td>
<td>110</td>
<td>260</td>
<td>5.2</td>
<td>19.8</td>
<td>2568</td>
<td>1179</td>
<td>15.5</td>
<td>1.56</td>
<td>3720.6</td>
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</tr>
<tr>
<td>B3</td>
<td>85</td>
<td>330</td>
<td>4.2</td>
<td>21.2</td>
<td>4828</td>
<td>264</td>
<td>19.1</td>
<td>1.91</td>
<td>4441.9</td>
<td>Flexure - Torsion</td>
</tr>
<tr>
<td>B4</td>
<td>90</td>
<td>250</td>
<td>5.5</td>
<td>18.6</td>
<td>2158</td>
<td>493</td>
<td>13.4</td>
<td>N/A</td>
<td>2678.8</td>
<td>Torsion</td>
</tr>
<tr>
<td>B5</td>
<td>90</td>
<td>290</td>
<td>2.8</td>
<td>20.8</td>
<td>2481</td>
<td>333</td>
<td>18.6</td>
<td>1.88</td>
<td>4241.6</td>
<td>Flexure - Torsion</td>
</tr>
<tr>
<td>B6</td>
<td>90</td>
<td>220</td>
<td>4.9</td>
<td>16.5</td>
<td>2033</td>
<td>693</td>
<td>15.2</td>
<td>N/A</td>
<td>2214.1</td>
<td>Torsion</td>
</tr>
<tr>
<td>B7</td>
<td>90</td>
<td>250</td>
<td>4.2</td>
<td>14.1</td>
<td>2460</td>
<td>391</td>
<td>20.0</td>
<td>1.00</td>
<td>2015.0</td>
<td>Torsion</td>
</tr>
</tbody>
</table>

P<sub>c</sub>: Load at first crack; P<sub>u</sub>: Ultimate Load;  
Δ<sub>cr</sub>: Deflection at first crack load; Δ<sub>uf</sub>: Deflection at ultimate load;  
ε<sub>cr</sub>: Strain in longitudinal reinforcement at failure; ε<sub>u</sub>: Strain in stirrups at failure.

### Cracking Patterns

The behavior of all specimens was observed under loading increments until failure and the cracking patterns were detected and marked. For ease of reference the notation front face was used for the beam side closer to the load as mentioned before. Figure 5 shows the cracking pattern after failure for the control beam B1 where the applied load was at the centerline of the specimen without any eccentricity. The first crack appeared at the center line of the
specimen on both the front and the back sides at approximately the same time and it was a flexure vertical crack. At higher loading levels, the number of cracks increased, their widths increased, and they extended from the front side towards the back side through the bottom face until they were connected. There were no cracks observed on the upper or left and right sides. The final cracks at the front and back sides of the beam followed a similar pattern as shown in Figure 5a and 5c. This is in agreement with the beam having zero load eccentricity. Finally, the beam failed in flexure due to the excessive increase in flexure cracks.

The rest of the specimens B2 to B7 showed a different cracking pattern from B1. The six beams generally had similar crack patterns behavior. The typical final cracking pattern is shown in Figure 6 with Beam B2 shown as an example. The first crack observed was a vertical flexure crack that began to appear at the middle third of the specimen at the front side followed by the same type of crack at the back side. After that, these cracks started to extend horizontally at the bottom face of the specimens to be connected at the middle third region. At higher loading levels, some cracks inclined at 45° began to appear on the front face that extended to the bottom face near the support, then some inclined and horizontal cracks appeared on the top face near the supports that extended to both the left and right sides forming again cracks inclined at 45°. It can be seen from Figure 2a and 2b that the front face showed clearly inclined shear cracks while the back face had vertical widely uniformly spaced cracks. At the end of loading, a bundle of cracks was formed at the support zone till failure. These cracking patterns well indicate the presence of torsional moments.

![Figure 5. Cracking pattern for specimen B1](image1)

![Figure 6. Cracking pattern for specimen B2](image2)
Figure 7 shows a comparison of the cracking pattern for the front face with the bottom one for Beams B2 to B7. As previously mentioned, these six beams showed a behavior typical for beams subjected to torsion. However, the effect of the parameters under study can be seen clearly in the variations between the beams. Comparing Beams B1, B2, and B3, as the load eccentricity increased, the number of cracks occurring in the middle third of the specimens decreased showing the least occurrence in Beam B2 with load eccentricity $e/B=0.333$. In addition, the diagonal cracks on the front face reached almost the full depth of the beams indicating torsional shear stresses and they were more pronounced in Beam B2. Beam B3 with eccentricity $e/B=0.167$ showed a crack pattern very similar to Beam B1 which was loaded without any eccentricity.

Beam B4 with the largest longitudinal spacing between stirrups (100 mm) showed more horizontal cracks on the bottom face than Beams B2 and B5 with spacing 75 and 50 mm, respectively. However, the inclined diagonal cracking patterns seen on the front face of the beams were different. In Beam B4, a clear distinct diagonal crack was seen on both sides of the span. While in Beam B2, a bundle of diagonal cracks was observed on both the left and right sides. As for Beam B5, the same two bundles were present however, the bundle width and the number of cracks within the bundle were higher than in case of Beam B2 and the cracks widths were lower. This larger number of cracks with smaller width in Beam B5 agrees well with it having the highest ductility factor among the three beams as shown in Table 2. Cracking pattern also showed dependency on the arrangement of the longitudinal reinforcement. Comparing the beams in Group (3), Beam B2 showed the largest number of diagonal cracks with the smallest widths followed by Beam B7 and then Beam B6 which showed two distinct diagonal cracks with larger width. This cracking pattern again correlates with Beam B2 showing the highest ductility behavior among the three beams.

The cracks on the back face for Beams B2, B4, B5 and B6 were vertical and rather widely dispersed with a uniform distribution along the length of the beam reaching only the mid depth of the beam. As for Beam B3 the cracks were also vertical but more closely dispersed and reaching almost the full length of the beam. The cracks at the back face for beam B7 were vertical in the middle third of the span and inclined towards the support at the two ends of the beam. These differences in the cracking patterns behaviors for the front and back faces confirm the occurrence of torsional stresses for beams B2 to B7.

**Load-Deflection Relationship**

The applied load versus the vertical deflection at mid span for all the beams is shown in Figures 8 through 10. Figure 8 shows the results for Group (1). Beam B1 showed a bilinear curve shape while Beams B2 and B3 exhibited a non-linear behavior until failure. The first crack load was almost similar for Beams B1 and B3 but slightly higher for
Beam B2. Also, the results show that Beam B3 had the maximum ultimate load of 330 kN while Beam B2 showed the minimum ultimate load of 260 kN while Beam B1 gave a value of 320 kN. However, these readings do not reflect the actual behavior since the loading for Beam B1 was terminated due to safety issues before the beam reached its ultimate capacity. The curve representing Beam B1 could progress as shown in Figure 8 by the dotted line extended from point A, which is the final test reading, to point B, which can be considered the predicted ultimate failure load. According to that, Beam B1 was supposed to show a higher load capacity which is supported by the FEM calculations that will be shown later. The readings of Beams B2 and B3 show that the bigger the eccentricity of the load the lower ultimate load capacity due to the higher torsional moments exerted on the specimen. Comparing the deflection at ultimate load, Beam B2 showed a slightly lower value compared to Beam B3.

Group (2) was tested to show the effect of longitudinal spacing between stirrups on the behavior of each specimen. The load-deflection curves for specimen B2, B4, and B5 are shown in Figure 9 where the three specimens showed initial linear behavior followed by nonlinear load-deflection behavior up to failure. A slight increase of 4% occurred when the spacing between stirrups was decreased from 100 to 75 mm. However, a noticeable increase of 16% was obtained when the spacing was decreased from 100 to 50 mm. As for the deflection at ultimate load, Beam B2 (75 mm spacing) showed a value of 19.8 mm, Beam B4 (100 mm spacing) shows minimum deflection of 18.6 mm while Beam B5 (50 mm spacing) showed the maximum deflection of 21.2 mm.

The load-deflection curves for Group (3) are shown in Figure 10. The three beams B2, B6 and B7 were designed to have the same bottom reinforcement ratio value of 0.78% but with different reinforcement arrangements as shown in Figure 2c, 2d and 2e where Beams B2 and B7 had uniform bar distribution but with different diameters while for Beam B7 the reinforced bars were concentrated on the side under the load. The values of the ultimate load obtained from the loading test were different despite having the same reinforcement ratios where Beam B2 showed the highest value of 260 kN followed by Beams B7 and then B6 gave the lowest value of 220 kN. To quantitatively explain this behavior, the values of the cracked moments of inertia (I) for the half section under load around the center line of the cross sections Y shown in Figure 2c, 2d and 2e, were calculated using equation 1. The values of the moment of inertia calculated were the highest value for Beam B2 and the lowest value for Beam B6. Comparing the values of the ultimate load for the corresponding beams, the values for the ultimate load follow the same pattern as shown in Figure 11. This means that behavior of the specimens is affected by the reinforcement distribution even if the reinforcement ratio is kept constant with the ultimate load value increasing as the amount of inertia increases.

\[
I = \gamma (\pi D^2)/4, \quad (\sum r_i^2) = K D^2, \quad (\sum r_i^2)
\]

Where \(K = \gamma \pi /4\)

I: Cracked moments of inertia  \(\gamma\): density of steel, \(D\): diameter of the reinforcing bar, \(r_i\): distance measured from the center line of the steel bar to the center line of the cross section of the beam.

**Strain in Stirrups**

The values of the strain in the stirrups for the seven beams are shown in Table 2. Beam B2 had the maximum strain value of approximately 1180 micro-strain while the other six beams gave strain values ranging between 132 to 694 micro-strain. The strain measured in stirrups shows some discrepancies. This can be explained since the local strain in the leg of the stirrup is greatly affected by its location with respect to the cracks. For each beam, a strain gauge was located at \(d/2\) from the face of the support; however, this position relative to the cracks induced can be different from one specimen to the other and could not be unified. In general, the web reinforcement did not reach its yield limit as the yield strain for bars with diameter 6 mm steel is 1690 micro-strain.

**Strain in Longitudinal Reinforcement and Modes of Failure**

The load strain curves for the longitudinal reinforcement for all the tested beams are shown in Figures 12 through 14. The yield strain for steel bars diameter 16 mm and 12 mm is 2330 micro-strain and 2475 micro-strain, respectively. For Group (1) with different eccentricity ratios, the longitudinal reinforcement in the three beams B1, B2, and B3 reached the yield limit before concrete crushing at 250, 240, and 270 kN, respectively. It can be seen from Figure 12 that Beams B2 and B3 showed very similar load strain behavior, where the yielding of main reinforcement is very clear through a bilinear curve. This shows that beams having low eccentricity values exhibit behavior close to that of beams without any eccentricity. For Group (2), Beams B2 and B5 with lower spacing between stirrups reached the yield limit at 240 and 280 kN while the longitudinal reinforcement of B4 with spacing of 100 mm did not reach the yield limit. This shows that the lower spacing between stirrups leads to a better ductile behavior and mode of failure in addition to higher ultimate load values. For Group (3), where the longitudinal reinforcement distribution was studied, Beam B2 with uniform distribution and higher bar diameter giving higher cracked moment of inertia showed yielding in the main reinforcement. While for Beams B6 and B7, changing the reinforcement distribution to non-uniform or lowering the area of the individual bar diameter showed a rather non-ductile behavior with the bars in Beam B6 not reaching the yield limit and those in B7 almost reaching the yield strain with a value of 2460 micro-strain.
Figure 8. Load-Deflection curves for Group (1)

Figure 9. Load-Deflection curves for Group (2)

Figure 10. Load-Deflection curves for Group (3)

Figure 11. Effect of bottom reinforcement distribution

Figure 12. Load-Strain curves for longitudinal reinforcement – Group (1)

Figure 13. Load-Strain curves for longitudinal reinforcement – Group (2)
Beam B1 failed in flexure with the main reinforcement clearly reaching the yield strain as shown in Figure 12. As for beam B3, flexural failure was considered based on the shape of the load strain curve despite the shear torsional cracks observed. Beams B2, and B5 showed yielding in the main reinforcement just before failure leading to a combined flexure torsion failure mode while reinforcement in Beam B6 almost reached the yield strain at failure. On the other hand, the longitudinal reinforcement in Beams B4 and B6 did not reach the yield strain and failed due to torsional stresses. Beam B4 showed the most pronounced torsional shear failure where it failed suddenly, and the diagonal cracks were very dominant.

**Structural Behavior**

Stiffness is the ability of the specimens to resist the deformation in response to the applied force where the more flexible the specimens are, the less stiff they are [18]. The stiffness can be calculated as the slope of the linear part of the ascending branch of the load-deflection curve. While toughness is the ability of the material to absorb energy and deform plastically without any fracture, where toughness can be determined as the area under load-deflection curve up to specified level of failure [18].

Ductility is the ability of the material to undergo large deformation without rupture before failure, where ductility can be measured using several measures as displacement ductility factor, drift index and curvature-ductility factor [27]. Ductility of specimens was evaluated using the displacement ductility factor ($\mu_\Delta$) which is defined as the ratio of the mid span deflection at the ultimate load to the mid span deflection at the yield load.

It was noted that Beam B1 with zero eccentricity and 16 mm uniformly distributed longitudinal bars had the maximum stiffness of 29.4 kN/mm, while B4 with longitudinal spacing between stirrups of 100 mm, 200 mm eccentric load and 16 mm uniformly distributed longitudinal bars had the least stiffness of 13.4 kN/mm. As for toughness, B3 and B5 had the maximum values of toughness of 4440 and 4240 kN.mm respectively while B7 had the minimum value of toughness of 2010 kN.mm. B1 had the maximum ductility factor of 2.17. On the other hand, the rest of specimens showed a ductility factor that ranges from 1.00 to 1.91.

From Table 2, it can be seen that decreasing the eccentricity of load from 200 mm (B2) to zero (B1) for Group (1) improved the ductility by 39.1% and the stiffness by 89.6% while the toughness decreased by 7.9%. The decrease in the longitudinal spacing between stirrups from 100 mm (B4) to 50 mm (B5) lead to an increase in the stiffness by approximately 39%. The decrease in the spacing between stirrups also lead to a more ductile behavior and an increase in the toughness by 58.3% where this can be explained by the ability of a member to resist fracture due to the presence of stirrups with small spacing in the longitudinal direction. Finally, changing the distribution of longitudinal steel bars from uniform distribution to non-uniform concentrated distribution under load for Beams B6 and B7 increased stiffness by 31.5% and decreased the toughness approximately by 9%. Both beams showed a lower ductile behavior compared to B2.

**4. Finite Element Analysis**

The finite element analysis was conducted in two consecutive stages as shown in Figure 1. First, the seven tested specimens were modeled and analyzed using the finite element analysis program ANSYS V.19 [24]. Each element in the model was defined using a specified element type from the ANSYS library where concrete is defined using SOILD 65, steel reinforcement is defined using LINK 180 and steel plates are defined using SOILD 45 [24]. To ensure that
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the model is acting similar to the experimental specimens, the boundary conditions were applied to simulate the actual conditions of the experiment where the supports were modeled in such a way that fixed supports were created by defining the values of the displacement in X, Y, and Z directions equal to zero and the rotation in X, Y, and Z directions also equal to zero. In this part, the model is assessed and compared with the data obtained from the experimental program to assure the validity and soundness of the model used.

The second part consisted of an extended parametric study where the study of different parameters was conducted to further understand the behavior of wide shallow beams under eccentric loading. Seven more beams were analyzed with varying parameters such as the compressive strength of concrete, the main reinforcement ratio, the spacing between the stirrups, and the number of branches of the stirrups used.

4.1. Comparison between Experimental Results and FEM analysis

The comparison between the experimental results and FEM analysis was conducted through cracking patterns, modes of failure, cracking loads, ultimate loads, load deflection curves, and load strain curves for transverse and longitudinal reinforcement. All numerical results obtained through the finite element analysis were compared with the experimental data shown in section 3.2.

Table 3. Summary of the FEM results for beams B1 to B7

<table>
<thead>
<tr>
<th>Specimen</th>
<th>P'cr (kN)</th>
<th>Pu (kN)</th>
<th>∆'cr (mm)</th>
<th>∆'u (mm)</th>
<th>E' main (μ)</th>
<th>P'cr / Pu</th>
<th>∆'cr / ∆u</th>
<th>∆'u / ∆u</th>
<th>E' main / E' max</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>145</td>
<td>364.8</td>
<td>2.75</td>
<td>11.33</td>
<td>4323</td>
<td>1.80</td>
<td>1.14</td>
<td>2.5</td>
<td>0.78</td>
<td>0.90</td>
</tr>
<tr>
<td>B2</td>
<td>115</td>
<td>243.2</td>
<td>3.19</td>
<td>16.90</td>
<td>2407</td>
<td>1.04</td>
<td>0.94</td>
<td>0.61</td>
<td>0.85</td>
<td>0.93</td>
</tr>
<tr>
<td>B3</td>
<td>115</td>
<td>298.6</td>
<td>2.10</td>
<td>16.42</td>
<td>3549</td>
<td>1.35</td>
<td>0.90</td>
<td>0.50</td>
<td>0.77</td>
<td>0.74</td>
</tr>
<tr>
<td>Group (2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>91.3</td>
<td>204.5</td>
<td>3.26</td>
<td>18.15</td>
<td>2511</td>
<td>1.01</td>
<td>0.82</td>
<td>0.60</td>
<td>0.98</td>
<td>1.16</td>
</tr>
<tr>
<td>B5</td>
<td>115</td>
<td>305.9</td>
<td>3.45</td>
<td>20.96</td>
<td>3493</td>
<td>1.27</td>
<td>1.05</td>
<td>1.23</td>
<td>1.01</td>
<td>1.41</td>
</tr>
<tr>
<td>Group (3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B6</td>
<td>95</td>
<td>194.9</td>
<td>3.46</td>
<td>13.60</td>
<td>1474</td>
<td>1.06</td>
<td>0.89</td>
<td>0.64</td>
<td>0.82</td>
<td>0.73</td>
</tr>
<tr>
<td>B7</td>
<td>95</td>
<td>264.0</td>
<td>2.78</td>
<td>17.42</td>
<td>2204</td>
<td>1.06</td>
<td>1.06</td>
<td>0.66</td>
<td>1.24</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Figure 15. Comparing cracking patterns for specimen B1

Figure 16. Comparing cracking patterns for specimen B2
Figure 17. Deformation for specimen B1 - FEM Analysis

Figure 18. Deformation for specimen B2 - FEM Analysis

Figure 19. Load deflection curves from experimental and FEM analysis - Group (1)

Figure 20. Load deflection curves from experimental and FEM analysis - Group (2)
Table 3 shows the output data for the seven specimens using the ANSYS program [24]. The analytical results gave good correlations with the experimental ones for Beams B2 to B7. However, Beam B1 shows higher values for the cracking load and ultimate failure load compared to the experiment. This confirms what was mentioned before in section 2.2 that the experimental results for Beam B1 are underestimated due to some issues with loading at the time of the experiment. The variation between the ultimate load and deflection at ultimate load between both experiment and analysis was within ± 24%. It was observed in the FEM analysis results that the longitudinal reinforcement of Beams B1, B2, B3, B4 and B5 reached the yield limit before the concrete crushing while the longitudinal reinforcement of Beams B6 and B7 did not reach the yield limit. The transverse reinforcement in FEM results did not reach their yield limit. These results follow the same trend as in the experimental output except for Beam B4.

Figures 15 and 16 show the comparison between the cracking patterns at failure for Beams B1 and B2 obtained from the experiment and that obtained from the FEA for both the bottom and the front sides. The cracking patterns follow a very similar pattern between the experiment and the analysis. Figures 17 and 18 show the deformation shape for the two beams obtained from the analysis from two perspectives and the stress distribution across the whole beams. The two figures clearly show the difference between Beam B1 where the eccentricity was taken as zero and Beam B2 with eccentricity 200 mm (e/B = 0.333). The stresses in B1 are highest at the midspan of the beam and have uniform values along the cross section. While for Beam B2 the stresses are highest at the side closer to the load with the direction of stresses changing at the opposite side from the load. The load deflection curves showed good correlations between experimental output and FEA. The load deflection curves for Group (1) are shown in Figure 19 while those for groups (2) and (3) are shown in Figures 20 and 21, respectively.

4.2. Parametric Study

To further understand the behavior of wide shallow beams under eccentric loading, a parametric study was conducted using the Finite element analysis program ANSYS V.19 [24]. Seven more wide shallow beams having the same dimensions as that tested experimentally (L=1700, b=600, t=200, and d=180 mm) were analyzed. The study of the specimens was performed through four groups as shown in Table 4 with the FEM analysis of Beam B2 used as a reference. Group (4) was designed to study the effect of using concrete with higher compressive strengths. It consisted of two beams; B8 and B9, in addition to B2. Group (5) consisted of two beams to study the effect of the ratio of the bottom reinforcement. In this group the reinforcement ratio was changed from 0.74% in Beam B2 to 0.42% for Beam B10 and 0.29% for Beam B11. The value 0.29% was chosen as it is the minimum reinforcement ratio specified for beams by the ECP 203-2018 [25].

As for Group (6), it also consisted of two beams to study the effect of the spacing between stirrups in the longitudinal direction. It has been widely assumed in most of the design codes [25, 26, 29] that the concrete cross section of wide beams is large enough to satisfactorily withstand the shear stresses without shear reinforcement. Previous research [15] however proved that the use of shear reinforcement can be quite beneficial. In addition, recent research, [12] and [28], pointed out that it will be quite difficult to accurately assess the shear capacity of large, lightly reinforced wide members without web reinforcement. This part of the research aims to assess the use of the stirrups and their spacing in case of wide beams subjected to eccentricity in the loading conditions. Maximum values for the spacing between stirrups are set for narrow beams subjected to torsion as 200 mm. However, no reference data can be
found for wide beams subjected to torsion. The values of 50, 75, 100 mm were previously used for the beams in Group (2) and here two more beams with spacing 150 mm and 200 mm are added. Shear reinforcement spacing limitations along the beam length are provided in the design codes as mentioned above, however, few limits exist for appropriate spacing of stirrup legs across the transverse direction [7]. Group (7) – Beam B14 – is used to study the effect of the stirrups spacing in the transverse direction or the number of legs of the stirrups where two legged stirrups having a transverse spacing of 560 mm are used and compared to Beam B2 with four branches stirrups and a transverse spacing of 187 mm. Figure 22 and Table 4 show the details of the Beams B8 through B14.

![Figure 22. Specimen details for Beams B8 to B14](image)

**Table 4. Details of the beam specimens for the parametric study and analysis results**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dimensions</th>
<th>Bottom Reinforcement</th>
<th>Stirrups</th>
<th>fcu (MPa)</th>
<th>FEM analysis results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>600×200×1700 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ANSYS B2</td>
<td>600×200×1700 mm</td>
<td>Ø6 @ 200 mm, e = 0.33</td>
<td></td>
<td>75 30</td>
<td>243.2 16.90 2407</td>
</tr>
<tr>
<td>Group (4)</td>
<td>B8 600×200×1700 mm</td>
<td>Ø6 @ 187 mm, 4 branches</td>
<td></td>
<td>75 40</td>
<td>301.7 15.64 2922</td>
</tr>
<tr>
<td></td>
<td>B9 600×200×1700 mm</td>
<td>Ø6 @ 187 mm, 4 branches</td>
<td></td>
<td>75 50</td>
<td>352.1 16.25 3225</td>
</tr>
<tr>
<td>Group (5)</td>
<td>B10 600×200×1700 mm</td>
<td>Ø6 @ 187 mm, 4 branches</td>
<td></td>
<td>75 30</td>
<td>191.2 15.39 3918</td>
</tr>
<tr>
<td></td>
<td>B11 600×200×1700 mm</td>
<td>Ø6 @ 187 mm, 4 branches</td>
<td></td>
<td>75 30</td>
<td>171.0 11.43 3610</td>
</tr>
<tr>
<td>Group (6)</td>
<td>B12 600×200×1700 mm</td>
<td>Ø6 @ 560 mm, 2 branches</td>
<td></td>
<td>75 30</td>
<td>139.1 8.89 2549</td>
</tr>
<tr>
<td></td>
<td>B13 600×200×1700 mm</td>
<td>Ø6 @ 560 mm, 2 branches</td>
<td></td>
<td>75 30</td>
<td>172.0 14.29 3117</td>
</tr>
<tr>
<td>Group (7)</td>
<td>B14 600×200×1700 mm</td>
<td>Ø6 @ 560 mm, 2 branches</td>
<td></td>
<td>75 30</td>
<td>203.0 15.20 3202</td>
</tr>
</tbody>
</table>

fcu: compressive strength of concrete, \( P_u \): Ultimate load, \( \Delta_u \): Deflection at ultimate load; \( \varepsilon_{\text{main}} \): Strain in longitudinal reinforcement at failure.

**Effect of the Compressive Strength of Concrete – Group (4)**

The three beams in Group (4) were studied having concrete compressive strengths of 30, 40 and 50 MPa. These values represent the common range used in practice for normal strength concrete in addition to one specimen with high strength concrete through the 50 MPa mix. The load deflection curves for Group (4) are shown in Figure 23. Increasing the compressive strength led to a notable increase in the ultimate load capacity where the ultimate load was
increased by 24% when the compressive strength was increased from 30 MPa to 40 MPa and by 45% when the compressive strength was increased to 50 MPa. The increase in the ultimate load of the beams can be linearly related to the concrete compressive strength. However, further study is needed to support this output. No significant change was noticed in the values of deflection where the deflection at ultimate load for B8 and B9 was 15.64 and 16.25 mm, respectively as seen in Table 4.

**Effect of the Main Reinforcement Ratio - Group (5)**

Figure 24 gives the load deflection curves for Group (5). A reduction in the ultimate load capacity of 21% and 30% was seen when the reinforcement ratio was reduced to 0.42% and 0.29%, respectively. In addition, ductility was reduced with the decrease in the main reinforcement ratio as the values of the deflection at ultimate load decreased by 8% for Beam B10 and more notably for Beam B11 by 32%.

**Effect of the Longitudinal Spacing between Stirrups - Group (6)**

Group (6) is an extension of Group (2) where more values for the spacing between stirrups were studied. Previously spacing values of 50, 75, 100 mm were used. Here, two more values were added; 150 mm for B13 and 200 mm for B12. The load deflection curves can be seen in Figure 25. The curves for the beams in this group are plotted with the analysis output for Beam B4 and Beam B5 as well as Beam B2 for reference. As the longitudinal spacing between stirrups decreased, the ductility and the ultimate capacity of the beam increased. Figure 26 shows the values of the ultimate load against the ratio of s/d for the five beams where s denotes the longitudinal spacing between the stirrups and d is the depth of the beam. The ultimate load decreases sharply up to s/d = 0.5 while for values large than that the slope of the curve is reduced. It should be noted here that s/d = 0.5 is the limit set by most codes [25, 26] for the maximum spacing between stirrups. The curve plotted in Figure 26 supports this stipulation as the ultimate load capacity is largely reduced up to s/d = 0.5. Increasing s more than 0.5d could lead to excessive loss in the ultimate capacity of the beam.
Effect of the Stirrups Leg Spacing (Transverse Spacing) - Group (7)

Group (7) aims to study the effect of the transverse spacing of the stirrups. Design codes have different guidelines for the limits for the transverse spacing. Eurocode 2 [29] puts a limit of 0.75 d or 600 mm which is the same as that for the longitudinal spacing while ECP 203-2018 [25] has a maximum limit of 250 mm. On the other hand, the ACI 318-19 [26] gives no limit for the value of the transverse spacing. The load deflection curves for Group (7) are shown in Figure 27. Beam B14 is compared with Beam B2 where the number of branches of stirrups was changed from 4 to 2 corresponding to a change of leg spacing from 187 mm to 560 mm. A reduction in the ultimate load capacity of 16.5% was seen and the mid span deflection was reduced by 10%.

5. Conclusions

A combined bending torsion experiment on wide shallow RC beam specimens was conducted. This paper presents the details of the experiment and the experimental results for seven beam specimens under eccentric loading. In addition, the experimental results were compared with the analytical results obtained using the finite element program ANSYS 19 followed by an extended analytical study to investigate the effect of various parameters on the behavior of wide shallow beams under eccentric loading.

Based on the results obtained in this research, the following points can be concluded:

- Applying vertical load with varying eccentricities to RC wide beams exerts torsional moments in addition to the bending and shear stresses which can be clearly seen in the cracking patterns and failure modes.
- Increasing the load eccentricity from 100 to 200 mm (Approx. 17 to 33% of the beam width) leads to a decrease in ultimate load capacity. Based on the results of the experimental program a decrease of 21% was obtained. In addition, a decrease in the stiffness and toughness of 19 and 16% was obtained due to an increase in the torsional stresses along the beam cross section.
- Beams with small load eccentricity (0.167) behave in a way similar to beams without any load eccentricity.
- Decreasing the spacing between stirrups helps in controlling the cracks width and increasing the ultimate load. The ultimate load capacity was almost doubled when the spacing to depth ratio (s/d) was decreased from 1 to 0.28. Ductility is also improved with the lower longitudinal spacing between stirrups.
- The arrangement and distribution of longitudinal steel reinforcement affects the behavior of the wide beams even when having the same reinforcement ratio. It can be rationalized that the cracking moment of inertia is the main factor governing the effect of the reinforcement distribution.
- Using uniform distribution of longitudinal bars with a large diameter leads to an improved behavior compared to using a small diameter keeping the same reinforcement ratio of steel. However, concentration of longitudinal reinforcement under the applied load increases the ultimate load capacity but decreases the ductility of the specimen.
- The modeling of the wide RC beams under eccentric loading using the finite element program ANSYS proved to be effective where good agreement was found between the output of the analytical program and the experimental results.
Using high strength concrete increases the torsional capacity of wide beams and increases the ultimate load capacity. An increase of approximately 25% was obtained when compressive strength was increased to 40 N/mm² from the original 30 N/mm² and of approximately 46% for compressive strength of 50 N/mm².

Decreasing the ratio of the main longitudinal reinforcement leads to a noticeable decrease in ultimate load capacity and the ultimate deflection.

The transverse spacing between stirrups legs across the width of the wide beam affects the beam behavior. This was noticed when changing the shape of transverse reinforcement from four branches to two branches thus leading to a decrease in the torsional resistance and the ultimate load capacity by approximately 16.5%.

6. Declarations

6.1. Author Contributions

The main ideas and the methodology of the research were discussed and decided by all authors. The manuscript was written by S.M. and R.M. and review was done by M.K. The results, discussions, interpretation, and conclusion were completed by all authors. 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


[26] ACI Committee 318. “Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (318R-19).” American Concrete Institute, (2019).


Evaluation of Parking Demand and Future Requirement in the Urban Area

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Abstract

Whatever vehicle is traveling, it needs to stop in order to arrive road users their different goals. In most universities, parking becomes an important campus resource, for being as a place to come frequently and to spend long period. Now days parking problems increase with repaid growth of car ownership. So traffic and parking impact can be consider as a major source of contention within any community and can raise additional costs for universities, as well as urban areas facilities. The study aims to evaluate the current parking situation on the university campus in terms of the available supply and required demand of parking spaces in order to recommend future parking spaces need for the next five years. Data has had been collected according to field traffic and engineering survey, Videography method was used for this purpose. Inventories, Intervi ews and questionnaires included. Data analysis conducted with the aided of AASHTO and equation methods. The study concluded future parking required is 140 vehicle-spaces for the year 2026, according to population rate of growth also illegal parking leads to interference with the movements of pedestrians and their crossing, as well as reducing the capacity of the roads in the study area.

Keywords: Parking; Parking Accumulations; Parking Demand; Urban Area.

1. Introduction

Most of the large and medium cities, as well as large educational areas such as universities and research centers suffer from the problem of parking availability. Due to the lack of planning and not given enough importance in master plan this problem appears clearly according to the mounting growth with rapid increase of Car Ownership matched by the growth in demand for stopped in suitable parking vehicles. When determining the width of a parking lane, keep in mind that it will likely be used as a traffic lane in the future, either constantly or throughout peak times [1]. In the city of Zurich, a study was conducted to find the best way to operate the mobility and parking systems on demand. The study found that the system can function successfully even when there are a limited number of parking spaces available, and that expanding capacity of parking by one parking place per automobile cannot provide any advantages. Furthermore, it ensures optimum parking spaces’ distribution for a certain on-demands and city transportations system [2].

The major objective of the University Tanage National case research depending project had been on the demand and supply car-parking facilities for students in college of engineering. Depending on the results of the parking interview, 88 percent of students reported to have parking problems. Nevertheless, in the assessment of current
parking supply, this scenario was the polar opposite, with just 85 percent of parking spaces available. The parking load is 89 percent, indicating there was a parking issue [3]. The study used two direct parking surveys, which will provide the latest reliable information as well as in developing parking policies in the capital [4]. A research was performed on the major campus parking owing to an analysis of projected parking demand, compared to the continuous increase in vehicle ownership, and exceeded the existing parking supply at Al- Mustansiriya University. Depending on the predictive algorithms, 644 more parking spaces will be required by the targeted year 2015 to meet the study area's projected parking demand [5].

Parking studies in Delhi were conducted in nine sectors, including business and retail locations. The peak parking saturation for cars and bikes is 3.25 and 6.21, respectively, indicating a significant spillover situation. The study conducted according to parking characteristics analysis it was suggested optimal utilization of available space as well as the study useful for engineers, planners and decision makers [6]. The goal of parking lot layout is to optimize the total parking spaces' number in the given area by taking three factors into account. First and foremost, the parking plan must allow for a constant flow of vehicles across the parking lots. Secondly, the designed to allow for safe pedestrian circulation from parking to building, as well as suitable parking space planting without interfering with sites illumination [7].

Utilizing the average, Fully Time Equivalents (FTE) staff number are projected to be approximately 625,000 and student numbers are anticipated to be over 2,800,000 throughout the UK Higher and Further Education sector, according to the British Parking Associations. The average result for vehicle parking bays at the UK University provides an estimation of 270,000 bays [8]. Excessive parking has a price tag attached to it. Parking garage development seems to have a high financial cost, as well as substantial opportunity costs. In several mixed-use areas, space is at a premium, and every parking space or garage signifies a lost chance for more stores, restaurants, and residences [9]. The main objective of this research is study the current situation of parking on the university campus in terms of the available supply and required demand of parking spaces. Evaluate the current situation by following the approved technical standards in order to recommend future parking spaces need for the next five years with aid of inventories, Interviews and questionnaires, taking into account the growth of the university and the random increase in the level of car ownership.

2. Literature Review

Several studies and researches have had been conducted for parking lots in terms of their capacity, demand and supply for parking in universities and urban areas, which are summarized as follows:

A research of parking supply and demand was performed at Shahid Bahonar University of Kerman (SBUK) in Kerman, Iran, utilizing a suggested technique to improve parking pays management for university employees and students. The research found that the suggested approach may assist decrease the time spent walking about looking for a suitable parking pays for both students and employees. Furthermore, the suggested application has the potential to improve employees and students satisfaction with parking management [10]. Two studies were conducted in the cities of Al-Hilla and Najaf in Iraq, the first one dealt with the off-street parking and the second with the on-street parking. According to the first research, average off-street parking turnover varies from 1.9 to 2.6, indicating poor use of each pays in the city center. The second discovered that most parked cars in both locations had a 30-minute waiting time, demonstrating that unlawful parking seems to be prevalent in the research region city on both weekends and weekdays [11, 12].

University of Kufa study parking demand in the university campus was suggested the required additional parking spaces are 260 and using the smart parks give encouraging results [13]. The study in university of Nizwa showed that the current demand for parking is more than the currently available parking spaces, and people may park their cars in illegal places. Based on the estimated demand, the required parking spaces for both students and staff have been calculated, taking into account improvement plans for the new campus [14]. The distribution of parking is insufficient, and there are irregular between supply and demand, as well as the parking duration highly and illegally [15].

In this study, parking data including parking duration, loads, and accumulation were used to analyze sub zones. After a thorough examination of the data, the research determines the parking issue for this subzone [16]. The study of Texas A&M Transportation Institute in downtown Houston, shown that the introduction of a SFpark-style smart parking system will save about 200,000 hours per year in decreases congestion delays while saving the city $4.4 million annually in congestion costs. Also Daily occupancy between (53-80)% was noted which have a significant spatial imbalance in the parking offered compared to the demand, meanwhile the parking oversupply may increase further if an intelligent parking system that focuses on efficiency and spatial distribution using dynamic pricing is introduced [17].

Previous studies and research that dealt with the demand for parking, which is actually available, showed that there is an increasing demand for parking, which is offset by a shortage of parking spaces in the urban area. Therefore, our study is one of the important research that addresses the increasing demand for parking, and it helps decision-makers
to predict the actual need to parking spaces for the next five years, based on mathematical models and according to the approved technical specifications.

3. Materials and Methods

Due to the rapid changes in the economic and social aspect that directly affected ownership in the Babylon province, this accompanied by a growing demand for parking in the city in general and the University of Babylon in particular, as it represents a center of attraction for trips different types. The parking supply available at the university provides 250 parking spaces approximately, distributed according to the master plan into one main and another four secondary parking lots. Study problem is summarizing as follow:

- The lack of an efficient and adequate program that adopts the process of defining and distributing the parking areas, according to the different objectives of the vehicle users on and off campus.
- The aggravation of random and illegal parking of vehicles and their failure to stop in designated places led to reduce the roadways capacity, as well as cases of overtaking on sidewalks, roads and various land uses.

The study used the scientific methodology in research, data collection and analysis to reach logical results, as shown in Figure 1. The campus is located 10 km south of Hilla city. Arrival and departure trips use Hilla -Najaf multi-lane roadways and (80- street) major arterial. As shown in Figure 2.

![Flowchart Represent the Research Methodology](image-url)
Data was calm depending on field survey traffic information and engineering survey included within A.M. and P.M. Peaking Hours Volume (PHV). Video technique method used in order to collect parked vehicles with the aid of questionnaire and interview to assess the current situation then suggest the appropriate development. Data listed in Tables and Figures. The study chose three parking lots within university of Babylon master plan, they are:
- Parking no. 1 located in the south gate of university campus
- Parking no. 2 located in front of faculty of Law.
- Parking no. 3 located in front of College of Science.
- Parking no. 4 located in the College of the Basic Education

The layout for each parking represented in Figures 3 to 5.
Figure 5. Illustrated the Layout of Parking No. 3 & 4 within Study Area

Table 1 illustrates the characteristics of parking; While Table 2 and Figure 2 explain the calculation of Parking Accumulation for Parking No. 1

<table>
<thead>
<tr>
<th>Parking Site &amp; No.</th>
<th>Type</th>
<th>No. of Bays (Veh.)</th>
<th>Parking Users</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking No. 1</td>
<td>Public</td>
<td>156</td>
<td>All</td>
</tr>
<tr>
<td>Parking No. 2</td>
<td>Public</td>
<td>116</td>
<td>Staff</td>
</tr>
<tr>
<td>Parking No. 3</td>
<td>Public</td>
<td>68</td>
<td>Staff and Faculty</td>
</tr>
<tr>
<td>Parking No. 4</td>
<td>Public</td>
<td>68</td>
<td>Staff and Faculty</td>
</tr>
</tbody>
</table>

Table 2. Represent Parking Accumulative for Parking no. 1.

<table>
<thead>
<tr>
<th>Time</th>
<th>In</th>
<th>Out</th>
<th>Parking Accumulative</th>
<th>No. of Bays</th>
</tr>
</thead>
<tbody>
<tr>
<td>07:30 - 08:00</td>
<td>6</td>
<td>2</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>08:00 - 08:30</td>
<td>12</td>
<td>1</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>08:30 - 09:00</td>
<td>25</td>
<td>2</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>09:00 - 09:30</td>
<td>22</td>
<td>2</td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>09:30 - 10:00</td>
<td>18</td>
<td>2</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>10:00 - 10:30</td>
<td>16</td>
<td>4</td>
<td>91</td>
<td></td>
</tr>
<tr>
<td>10:30 - 11:00</td>
<td>18</td>
<td>6</td>
<td>102</td>
<td></td>
</tr>
<tr>
<td>11:00 - 11:30</td>
<td>18</td>
<td>5</td>
<td>115</td>
<td></td>
</tr>
<tr>
<td>11:30 - 12:00</td>
<td>14</td>
<td>3</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>12:00 - 12:30</td>
<td>8</td>
<td>12</td>
<td>120</td>
<td>156</td>
</tr>
<tr>
<td>12:30 - 01:00</td>
<td>4</td>
<td>17</td>
<td>108</td>
<td></td>
</tr>
<tr>
<td>01:00 - 01:30</td>
<td>6</td>
<td>21</td>
<td>92</td>
<td></td>
</tr>
<tr>
<td>01:30 - 02:00</td>
<td>5</td>
<td>24</td>
<td>73</td>
<td></td>
</tr>
<tr>
<td>02:00 - 02:30</td>
<td>3</td>
<td>30</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>02:30 - 03:00</td>
<td>13</td>
<td>7</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>03:00 - 03:30</td>
<td>8</td>
<td>6</td>
<td>55</td>
<td></td>
</tr>
<tr>
<td>03:30 - 04:00</td>
<td>6</td>
<td>14</td>
<td>47</td>
<td></td>
</tr>
<tr>
<td>04:00 - 04:30</td>
<td>0</td>
<td>17</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>04:30 - 05:00</td>
<td>0</td>
<td>23</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>05:00 - 05:30</td>
<td>0</td>
<td>3</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>
Traffic Data for parking observed from 07:30 A.M - 05:30 P.M was noted. Maximum accumulation appeared at 11:30 A.M when the No. of Bays equal to 156 vehicles. The initial found = 5 vehicle, peak hour period accrue from 11:00 A.M. to 12:00 P.M. The parking demand decrease respectively until 05:30 P.M. At the end period of data collection, the same method has used in order to calculate the parking parameters, as shown in Table 3 and Figure 6.

![Figure 6. Represent Parking Accumulation for Parking No. 1](image1)

### Table 3. Characteristics of Parking According to Peak Hour in the Study Area

<table>
<thead>
<tr>
<th>Parking No.</th>
<th>No. of Bays</th>
<th>Peak hour period</th>
<th>Data collection period</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2</td>
<td>116</td>
<td>10:15 - 11:15 A.M. 01:45 - 02:45 P.M.</td>
<td>07:15 A.M. - 05:45 P.M.</td>
</tr>
<tr>
<td>P3</td>
<td>68</td>
<td>12:15 - 01:15 P.M.</td>
<td>07:15 A.M. - 05:15 P.M.</td>
</tr>
<tr>
<td>P4</td>
<td>68</td>
<td>11:45 A.M. - 12:45 P.M.</td>
<td>07:15 A.M. - 05:45 P.M.</td>
</tr>
</tbody>
</table>

![Figure 7. Represent Parking Accumulation for Parking (1, 2, 3)](image2)
From Figure 7 the distribution of parked vehicle for parking No. 2, parking accumulation at 07:15 A.M, the initial found is 5 vehicle, then its increase respectively until reaching the A.M peak hour (10:15 A.M to 11:15 A.M) then the parked vehicle increase within P.M peak hour (01:45 P.M to 02:45 P.M). The distribution of Park vehicle decreases until reach at 05:45 P.M the end of data collection period. The study observed that the distribution of parked vehicle for parking No. 3, the parking accumulation at 07:15 A.M, the initial found is 5 vehicle, Then it’s increase respectively until reaching the peak hour (12:15 to 01:15 P.M) the distribution of Park vehicle decreases till reach at 05:45 P.M the end of data collection period.

Finally the distribution of parked vehicle for parking No. 4, The parking accumulation started at 7:15 A.M. the initial found is 5 vehicle, Then it’s increase respectively until reaching the peak hour (11:45 to 12:45 P.M) the distribution of Park vehicle decreases till reach at 05:45 P.M. the end of data collection period. The Maximum parking accumulation for all parking lot in within study area equal to (582) vehicle.

3.1. Parking Duration

Due to traffic and exploratory survey within study area, the duration period in the parking of the university campus was determined as show in Table 4.

<table>
<thead>
<tr>
<th>Parking No.</th>
<th>Duration of parking Period</th>
<th>Percentage of parked vehicles (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15 min.</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>15 - 60 min.</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>1 - 4 hr.</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>&gt; 4 hr.</td>
<td>43</td>
</tr>
<tr>
<td>2</td>
<td>15 min.</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>15 - 60 min.</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>1 - 4 hr.</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>&gt;4 hr.</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>15 min.</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>15 - 60 min.</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>1 - 4 hr.</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>&gt; 4 hr.</td>
<td>52</td>
</tr>
<tr>
<td>4</td>
<td>15 min.</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>15 - 60 min.</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>1.0 - 4 hr.</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>&gt; 4 hr.</td>
<td>16</td>
</tr>
</tbody>
</table>

From Table 4, it is clear that the percentage of parked vehicles (1-4 hr.) and (>4 hr.) is higher than the rest, which indicated significant impact on demand for parking in the study area.

3.2. Walking Distance

Walking distance one of the indicative indicators of the parking efficiency, In addition, it contributes to reduce illegally parking, which causes conflict points in movements between vehicles and pedestrians. The average walking distance for the study parking was (189) m, which is considered as average shortest distance between pedestrian parking exit and the entrance of nearest building in study parking facilities.

4. Data Analysis, Result and Discussions

For determining the demand for parking within selected study area, data analyzed with the aided of criteria and equations described in Table 5:
Table 5. Parking Demand for Park no.1

<table>
<thead>
<tr>
<th>Parking Facilities</th>
<th>Description</th>
<th>Parking Facilities</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of bays</td>
<td>156</td>
<td>Commuters now being served</td>
<td>0.70x201x4=563 (space-hr.)</td>
</tr>
<tr>
<td>No. of vehicle parked daily</td>
<td>201</td>
<td>visitors now being served</td>
<td>0.3x201x2=121 (space-hr.)</td>
</tr>
<tr>
<td>Commuters parked %</td>
<td>70</td>
<td>Total (space- hour) served</td>
<td>563+121=684</td>
</tr>
<tr>
<td>Average commutrs parked (hr.)</td>
<td>4</td>
<td>Total No. of vehicles turned away</td>
<td>18x4= 72</td>
</tr>
<tr>
<td>Visitor pared %</td>
<td>30</td>
<td>No. of (space – hr.) demand</td>
<td>684+72=756</td>
</tr>
<tr>
<td>Average visitor pared (hr.)</td>
<td>2</td>
<td>No. of parking space available</td>
<td>756-684=72</td>
</tr>
<tr>
<td>No. of (space) required</td>
<td>17</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To determine number of space required for each parking, the study use Equation 1 [18].

\[ S = \sum_{i=1}^{N} (t_i) \]  

(1)

where; \( S \) = practical numbers of space-hrs. of supply for a specific time, \( N \) = available numbers of parking pays, \( t_i \) = total time’s length in hrs. once the \( i \)th pay could be legally parked throughout the specific time, \( f \) = efficiency factor. Recommended \( f \) value is (85)% for surface lots \((0.85\times 5\times N= 17)\), and \( N = 33 \) No. of parking space required.

In the same way, the calculated parking parameters in order to obtain the supply and required parking spaces. As Table 6;

Table 6. Represent Calculated Parking Parameters in the study area

<table>
<thead>
<tr>
<th>Parking No.</th>
<th>No. of bays</th>
<th>No. of park vehicle daily</th>
<th>Commuters now being served</th>
<th>Visitors now being served</th>
<th>Total (space-hour) served</th>
<th>Total No. of vehicles turned away</th>
<th>No. of (space-hr.) demand</th>
<th>No. of parking space available</th>
<th>No. of parking space required</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>116</td>
<td>169</td>
<td>520</td>
<td>78</td>
<td>598</td>
<td>40</td>
<td>638</td>
<td>40</td>
<td>9</td>
</tr>
<tr>
<td>3</td>
<td>68</td>
<td>102</td>
<td>330</td>
<td>39</td>
<td>369</td>
<td>24</td>
<td>393</td>
<td>24</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>68</td>
<td>110</td>
<td>326</td>
<td>57</td>
<td>383</td>
<td>32</td>
<td>383</td>
<td>10</td>
<td>3</td>
</tr>
</tbody>
</table>

4.1. Current Parking Demand and Future Forecast

In order to determine the demand for parking in relation to the current situation and future forecasting, in the study area the following factors studied, as shown in Table 7.

Table 7. Parking Demand and Future Forecasting within Study Area

<table>
<thead>
<tr>
<th>Required Parking Space</th>
<th>Illegally Parked Vehicles</th>
<th>Total places Required for Parking Vehicles</th>
<th>No. of Parking Bays</th>
<th>Car Ownership Rate of Increase</th>
<th>Number of Campus Members (Study Year 2021)</th>
<th>Number of Campus Members (Target Year-2026)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>69</td>
<td>104</td>
<td>408</td>
<td>3%</td>
<td>22984</td>
<td>26645</td>
</tr>
</tbody>
</table>

From observations, inventories available and the questionnaire form prepare in the study area, It was observed that (11.70) percent of the campus residents use the personal vehicle (PCU) as a means to arrive their goals, as shown in Equation: \( 22984 \times 0.117 = 2689 \).

Assuming the annual car ownership for the campus population is 6 percent. This is consistent with the previous studies that were conducted, as well as the nature of the economic and social conditions that surround the city.

\[(1 + r)^n = (1 + 0.06)^5 = 1.338 \]

1.338 \( \times 2689 = 3599 \)

Future forecasting parking required (for next 5 years).

\[
\frac{\text{future need on parking spaces}}{\text{No. of expected vehicle for the next 5 years}} = \frac{\text{Current demand for parking}}{\text{No. of vehicles in the study year}} \]

\[ \frac{X}{3599} = \frac{104}{2689} = \text{Future need on Parking spaces} = 140 \]

These results have been reached based on a comprehensive field traffic study in order to calculate expected number of parks vehicle daily and total space hour serviced to get number of parking space available and required. Then use a mathematical model to determine the number of spaces that are expected and needed for the target year 2026, which
agree with specification standard.

5. Conclusions

The study reached many conclusions and recommendations summarized as follows:

- The future parking required are 140 vehicle-space for the year 2026, according to university campus population rate of growth, based on analytical study of traffic data.
- The study monitored through observations and questionnaire that (4 hr.) Parking is the average parked vehicles in the study area.
- Parking accumulation accessed peak period at (11:00 A.M. to 12:00 P.M.), (10:15 - 11:15 A.M. and 01:45 - 02:45 P.M.), (12:15 - 01:15 P.M.) and (11:45 A.M. - 12:45 P.M.) for parking no.1, 2, 3 and 4, respectively. While the maximum vehicle space accumulation for all study parks equal to (582).
- The study observed that there is an illegal parking of vehicles, which leads to interference with the movements of pedestrians and their crossing, as well as reducing the capacity of the roads in the study area.
- Equipping the parking with the necessary traffic signs, marking and lighting, will increase level of service and parking efficiency.
- This study, with a scientific and technical methodology, will help decision-makers to determine the priorities, including working to provide parking spaces for vehicles based on the different uses of the land according to the basic design of master plan within study area, safely, efficiently and conveniently.
- This study will encourage researchers and planners to carry out similar and comprehensive studies in urban areas, as well as providing a database that is compatible with the growth of parking demand, within a scientific planning that is consistent with university policies to avoid wasting financial and human resources.
- The study faced many challenges and difficulties in data collecting and analyzing especially during peak hour volume due to the conditions associated with the emergence of the Corona virus, which restricted researchers. Nevertheless, the study proceeded to provide the necessary data and samples to make it conform to the conditions of the study area.

6. Declarations

6.1. Author Contributions

A.N.A., A.R.I. A., and H.K.K.A. contributed to the design and implementation of the research, to the analysis of the results and to the writing of the manuscript. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

This data have been collected according to traffic and engineering survey with the aided of questionnaire and interviews with the approval of the College of Engineering - University of Babylon, Iraq.

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.

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A Case Study on The Mechanical and Durability Properties of a Concrete Using Recycled Aggregates

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Abstract

In Morocco, Recycled Aggregate Concrete (RAC) is not promoted unlike developed countries like France, Canada, US and many others. This article aims to present a Moroccan study related to the characterization of RAC and compare it with several studies all over the world. It focuses on compressive strength as the main mechanical characteristic and the porosity as the physical property that affects durability. The protocol is based on crushing concrete from demolished building and producing aggregates that are used in making experimental samples of RAC with different percentages of replacing Natural Aggregates (NA) by recycled ones. The first part of experimental study is to determine compressive strength of these samples after 7, 21, 28 and 90 days of confectioning it. Test results prove that above 25% of replacement level, the compression drops considerably and the Recycled Aggregates (RA) can’t replace the naturel ones. The second part of studies focuses on studying porosity as indicator of durability according to the performance approach. It concludes that the RAC may be used in a construction with a required life of 100 years specially building and roads. For high standards constructions or construction in a specific environment, more studies should be done.

Keywords: Concrete; Natural Aggregates; Recycled Aggregates; Replacement Percentage.

1. Introduction

Recycling aggregates is increasingly valuing in order to struggle with the shortage of natural resources and to comply with the sustainable development requirements. Even if there are no law or standards governing the use of RA in Morocco, several studies and researches are in progress to determine the influence of the replacement of NA by recycled ones on the concrete characteristics. This article, part of a doctoral study in civil department at Mohammadia School of engineers-Morocco, aims to characterize a concrete manufactured based on a crushed old concrete from a demolished building. The study of the concrete is based on two parameters:

- Compressive strength of concrete samples as an indicator of mechanical properties;
- Porosity as a physical parameter that influences durability based on the performance approach.

The main objective of the study is to determine the percentage of replacement of NA by recycled ones appropriate to an acceptable concrete from the technical point of view. The protocol of experimental tests will focus on studying the compressive strength of concrete samples at different ages and different percentage replacements than determining the amelioration on this mechanical property when using additives specially adjuvants and finally testing the porosity of concrete to conclude about its durability.

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2. Literature Review

The interest on recycled materials as a sustainable replacement of NA has grown this last decade due to their respect of environmental and reuse requirements. RAC is thus promoted as an innovative solution that combine innovation, respect of the environment and reuse of materials without harming technical and sustainability requirements [1-3]. Many studies have been done to determine its properties comparing to concrete using NA. A wide variability is found in mechanical and durability results and so the use of RAC is restricted to non-structural applications such as road sub-base materials and landfilling materials [4].

The study of RAC characteristics must be preceded by defining RA properties. Based on many studies, RA have higher water absorption, lower density, lower resistance to fragmentation, more pores and an irregular form compared to NA [4, 5]. This decrease in properties affects directly RAC properties specially with high replacement percentage of NA by RA.

The compressive strength is the most effective characteristic which affects all mechanical, durability and others properties [6, 7]. From a compressive strength point of view, using RA is accepted up to 20-30 % replacement of NA [8-11], 25% is the optimal replacement ratio according to Etxeberria et al. [11]. Above this value, the compressive drops considerably: 25% drop for 100% replacement by Bai et al. [6], 30-40% drop by Kazmi et al. depending on the quality of the original aggregates [5]. Even if the modulus of elasticity, tensile strength and flexural strength studies prove that their drops have almost the same trend as compressive strength results [7, 12, 13], the loss of compressive strength of concrete is more sensitive to the incorporation of RA than the others properties [6].

Durability performances: deformation (drying shrinkage), impermeability, chloride penetration resistance, carbonation resistance, frost resistance and alkali aggregate reaction of RAC are also weaker than NAC ones [3, 8, 9, 14]. Thomas et al. even noted that increases decrease with the increase of the degree of replacement for RAC [9]. The open porosity, sorptivity and rapid chloride permeability test values of RAC reach their maximum when percentage of NA replacement is 100% [14]. Density value decreases 5% for 20% replacement percentage of NA by RA and 20% for 100% replacement [9], Rao et al. reported 7.37% water absorption for concrete with 100% RA and 6.54% for concrete with 50% RA [3].

Some other researcher’s conclusions relative to RAC durability are: Impermeability is 2.47 times higher by immersion and 1.7 times by capillarity than that of a corresponding NAC specimen according to Guio et al. [15], Kou & Poon found 9.5% lower resistance to chloride permeability for 100% RCA mixture [10].

Many methods are proposed to improve RAC properties such as:

- Adjusting water-cement ratio [16].
- Removing adhered mortar. In fact, adhered mortar affects many properties specially durability and strength because of its high permeability [17].
- Adding pozzolanic materials or water repellent to improve cement matrix [18].
- Adding mineral admixtures such as fly ash and silica fume. The use of fly ash reduces the permeability and improves the workability of RAC. Silica fume, due to its small size and its large surface area, improves the microstructure of concrete creating a denser matrix [19].
- Using new method of mixing method of RAC such as two or three stage mixing methods [20].
- Using physical and/or chemical methods to enhance RAC without removing adhered mortar essentially encapsulation and impregnation by a foreign chemical [21]. These techniques may have inconvenient like reducing the compressive strength of RAC [17, 18, 22, 23].

3. Research Methodology

The experimental protocol aims to characterize the RAC and to determine the percentage of replacement of NA by recycled ones that respects the technical requirements. These tests also focus on enhancing the mechanical and durability properties of RAC.

3.1. Experimental Tests Related to Compressive Strength of RAC

The objective of the study is to determine the RAC compressive strength, considered as the main mechanical characteristic. The test is done according to the NF EN 12390-3 standard [24]. The concrete tested is a usual concrete, used in all types of building and roads without any specific requirements. It should have a compressive strength of 25 MPa after 28 days of confecting it and an average slump of 7-9 cm after mixing it.

The concrete formulation using dreux-gorisse method [25] is:
Table 1. The concrete formulation for 1 m³ of concrete

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Mass (kg)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>190</td>
<td>0.2</td>
</tr>
<tr>
<td>Cement</td>
<td>350</td>
<td>0.3</td>
</tr>
<tr>
<td>Sand</td>
<td>473</td>
<td>0.3</td>
</tr>
<tr>
<td>Gravels [5-12.5] mm</td>
<td>376</td>
<td>0.15</td>
</tr>
<tr>
<td>Gravels [12.5-31.5] mm</td>
<td>573</td>
<td>0.2</td>
</tr>
</tbody>
</table>

To determine the effect of the replacement percentage on concrete characteristics, several rates are proposed: 0, 25, 50, 75 and 100%. The choice of proportions is based on varying over the entire field from 0 to 100%. The Figure 1 shows steps of concrete preparation, specimen conservation and crushing.

Figure 1. Concrete specimens preparation steps: (a) Concrete and test tubes preparation; (b) Conservation of test specimens; (c) Crushing concrete tubes

In order to improve the compressive strength results for different replacement levels, the addition of adjuvants is tested. The chosen additives are plasticizers because of their capacity to reduce the water requirement of aggregates and thus improve the quality of the concrete. Three products from SIKA-MAROC are tested:

- Plasticizer (water reducer) named BV40.
- Superplasticizer or high-water reducer named Sika ViscoCrete Tempo 10M.

Adjuvants are used in replacement of cement. The ratio of replacement tested will be chosen between 0.5 and 1.5%.

3.2. Experimental Tests Related to RAC Durability

In order to validate the possibility of replacing an ordinary concrete by a RAC, additional technical studies must be carried out essentially a durability study. The technical validation of the RAC depends on several elements mainly the type of project, its location, external conditions and others. The performance approach is a new developed protocol that has been developed recently to characterize a concrete from a durability point of view [26]. This approach focuses on many criteria to choose parameters to study essentially the type of project which affects the required age. For an ordinary building or road, the required age is 50 years, it might reach 100 years for special buildings or engineering works. For civil developed projects like bridges, the required life exceeds 100 years.

The second parameter is environment type. It depends on the project locations (dry, wet, exposed to sea salts, etc.). For our project, we will focus on a building with a required life of maximum 100 years located in a usual environment.
Based on the performance approach, the requirements of the concrete to be durable are summarized in the table below.

<table>
<thead>
<tr>
<th>Porosity (%)</th>
<th>Building and civil engineering works</th>
<th>30-50 Building</th>
<th>&lt; 30 Building</th>
<th>Required life</th>
<th>Environment type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P &lt; 14</td>
<td>P &lt; 16</td>
<td>P &lt; 16</td>
<td>P &lt; 16</td>
<td>- Dry and very dry (RH &lt;65%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- Permanently wet (including immersion in fresh water)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wet, rarely dry (RH&gt; 80%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Exposure to sea salts or deicing, but not in direct contact with seawater with law (Cl-)</td>
<td></td>
</tr>
</tbody>
</table>

Porosity is, thus, the parameter to study to judge concrete’s durability. It is calculated according to NF P18-459 standard [27] using the following equation:

$$£ = \frac{(\text{M}_{\text{air}} - \text{M}_{\text{sec}})}{(\text{M}_{\text{air}} - \text{M}_{\text{eau}})} \times 100$$  \hspace{1cm} (1)

The test is based on the inhibition of the test sample in a constant pressure chamber. The test piece is then placed in a hydrostatic balance suspension system and its weight $\text{M}_{\text{eau}}$ is calculated. After removing the test tube from the water and wiping it quickly to remove surface water without removing water from the pores, $\text{M}_{\text{air}}$ is calculated as the weight of tube with intern water. At the end of the trial, the specimen is placed in the oven to remove the interstitial water and thus to determine $\text{M}_{\text{sec}}$ as the weight of solid phase only.

### 4. Results and Discussions

#### 4.1. Experimental Tests Related to Compressive Strength of RAC

The compressive strength tests are done with several percentages of replacement level: 0, 25, 50, 75 and 100% to define the drop evolution of strength based on the formulation used. The samples are crushed after 7, 21, 28 and 90 days of confectioning it. These days will show the evolution in time of the strength for every percentage of replacement. The table below resume the tests results.

<table>
<thead>
<tr>
<th>Percentage of replacement of NA by RA</th>
<th>Day of crushing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>0% RA (Reference)</td>
<td>17</td>
</tr>
<tr>
<td>25% RA</td>
<td>16</td>
</tr>
<tr>
<td>50% RA</td>
<td>15</td>
</tr>
<tr>
<td>75% RA</td>
<td>14</td>
</tr>
<tr>
<td>100% RA</td>
<td>14</td>
</tr>
</tbody>
</table>

To study the evolution of strength in time for every percentage level, a graph is established for every replacement level evolution between 7, 21, 28 and 28 days.

![Figure 2. Compressive strength for different percentage replacement after 7, 21, 28 and 90 days of confection](image-url)
For all replacement values, the compressive strength increases over time. The slope of evolution curve is different: For 0% replacement level for example, the compressive at 7 days is 57% of final strength at 90 days, for 25% it is 65% and above 50% of replacement level, we attend more than 75% of final compression at 7 days. The determination of the effect of replacement percentage in the compressive strength at several ages will be studied based on the graph below. The graph shows that, for all ages, the compressive strength drops with the increase of replacement level.

For 25% replacement level, the drop of compressive strength at 28 days is 8% comparing to the reference value (25Mpa) attended by the reference samples (0% RA), which makes this replacement level acceptable from a technical point of view. For 50% and more, the drop is more than 30%. The replacement isn’t authorized from a technical point of view. In order to improve these results, the ad of adjuvants is tested. The second section concerns then, compression tests on advanced concrete formulas. It aims to improve the strength of concrete for replacement percentages above 25%, ideally 50%. The tests concern the determination of an optimal combination between the choice of plasticizer to adopt and the percentage of replacement of the cement.

This phase is subdivided into 3 stages:

- Determination of the effect of adding an additive on compressive strength for various percent replacement.
- Use of several types of additives in order to conclude on the effect of the additive chosen.
- Testing several percentages of cement restitution by the admixture in order to choose the replacement percentage that gives the best result. The percentages to be tested were chosen by referring to the recommendations on each product sheets.

For the mixture with additives, we used three products from SIKA- MAROC:

- Plasticizer (water reducer) named BV40.
- Superplasticizer or high-water reducer named Sika ViscoCrete Tempo 10M.

**First Step: Determining the Effect of the Additive on the Compressive Strength of RAC with Different Percentage of RA**

Table 4 summarizes the results of compressive strength tests done for several test specimens with percentages covering the entire field from 0 to 100% with and without plasticizer. The test is done after 28 days of confection. The additive used is the plasticizer with 1% restitution of cement.

<table>
<thead>
<tr>
<th>Percentage of RA in RAC</th>
<th>Compressive strength at 28 days (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without plasticizer</td>
</tr>
<tr>
<td>25%</td>
<td>23</td>
</tr>
<tr>
<td>50%</td>
<td>18</td>
</tr>
<tr>
<td>75%</td>
<td>17</td>
</tr>
<tr>
<td>100%</td>
<td>15</td>
</tr>
</tbody>
</table>
These tests adhere to the results of concrete without additives where compression decreases with increasing replacement of NA by RA.

**Second Step: Choosing the Additive Giving the Best Strength**

This step consists of testing several generations of plasticizers. A plasticizer with an average water requirement reduction rate of 5 to 15%, a superplasticizer which can reduce water requirement by 30% and a new generation superplasticizer which in addition to its high reduction in water requirement, allows other more advanced performances: very long maintenance of maneuverability, pumping over long distances, adaptable with all the consistency of concrete. Tests are done on RAC with a percentage of replacement of 50% using 1% as a ratio of restitution of cement. Results are summarized in Table 5.

<table>
<thead>
<tr>
<th>Type of additive</th>
<th>Compressive strength at 28 days (MPa)</th>
<th>Percentage of amelioration of strength at 28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>18</td>
<td>-</td>
</tr>
<tr>
<td>BV40</td>
<td>23</td>
<td>25%</td>
</tr>
<tr>
<td>Tempo 10 M</td>
<td>21</td>
<td>19%</td>
</tr>
<tr>
<td>SFR</td>
<td>17</td>
<td>-08%</td>
</tr>
</tbody>
</table>

Compressive strength improves with the addition of the plasticizer as well as the superplasticizer. The new generation superplasticizer gives a lower result. This can be interpreted as a poor interaction with the aggregates and their attached cement paste. We can conclude that the quality of aggregates is conform to a plasticizer because of their low need of extra water to be 100% pre-hydrated.

**Third Step: Choosing the Optimal Percentage of Replacement of Cement that gives the Better Strength**

The effect of adding an adjuvant depends mainly on the rate of its use. As noted on the technical data sheet of the plasticizer [28], the dosage chosen should be between 0.2 and 1.5% of the binder weight. To choose the optimum value, comparison tests are carried out for three values: 0.5% as minimum value, 1.5% as maximum value and 1% as average value. The test results are summarized in the table 6 below. Notice that the plasticizer is the additive used and tests were done on RAC with 50% replacement ratio.

<table>
<thead>
<tr>
<th>Type of plasticizer</th>
<th>Percentage of the plasticizer</th>
<th>Compression at 28 days (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tempo 10M</td>
<td>0,5%</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>1,0%</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>1,5%</td>
<td>16</td>
</tr>
</tbody>
</table>

Tests conclude that 1% is the optimum value for restitution. Beyond this value the compression drops.

**4.2. Discussion about Mechanical Test Results**

These tests aim to characterize RAC from the compressive strength point of view and compare its characteristics with NAC ones. The first study phase demonstrate that the compression decreases with the increase of the restitution rate and that the optimum replacement percentage of NA by RA is around 25%. The compressive strength drop is insignificant (8%). This is in line with the conclusions of the majority of researches done in this direction that found an optimal replacement percentage between 20 and 30 %.

Beyond 50%, the compression drop exceeds 30% compared to NAC. Thus, above 50% as replacement level, the use of RA is not recommended in concrete. The second study phase concerns improving the quality of a concrete with 50% RA by adding an additive. Tests demonstrate that the use of plasticizer with a cement replacement rate of 1% is the ideal combination to improve strength. The drop improves from 30 to 8%. Researchers propose, apart from the addition of adjuvants, several methods of improving the compressive strength of concrete such as sorting demolished concrete and choosing the good quality only or removing adhered mortar [4].
4.3. Durability Characterization of RAC

As exposed above, the study of concrete durability will be done based on the porosity test results. The test was done respecting standards and the results are resumed in the Table 7 below. With £: The porosity of hardened concrete (%).

Table 7. The porosity experimental results

<table>
<thead>
<tr>
<th>Replacement percentage</th>
<th>$M_{se}$ (g)</th>
<th>$M_{air}$ (g)</th>
<th>$M_{eau}$ (g)</th>
<th>Porosity test results (%)</th>
<th>Porosity value chosen (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100%</td>
<td>2322.2</td>
<td>2464.1</td>
<td>1444.8</td>
<td>13.92</td>
<td>13.85</td>
</tr>
<tr>
<td></td>
<td>2318.7</td>
<td>2459.4</td>
<td>1442.5</td>
<td>13.83</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2310.3</td>
<td>2450.7</td>
<td>1434.8</td>
<td>13.82</td>
<td></td>
</tr>
<tr>
<td>75%</td>
<td>2324.5</td>
<td>2465.6</td>
<td>1430.1</td>
<td>13.63</td>
<td>13.59</td>
</tr>
<tr>
<td></td>
<td>2298.4</td>
<td>2439.9</td>
<td>1415.8</td>
<td>13.82</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2298.3</td>
<td>2434.8</td>
<td>1410.9</td>
<td>13.33</td>
<td></td>
</tr>
<tr>
<td>50%</td>
<td>2263</td>
<td>2423</td>
<td>1240.5</td>
<td>13.53</td>
<td>13.363</td>
</tr>
<tr>
<td></td>
<td>2271.6</td>
<td>2429.4</td>
<td>1247.7</td>
<td>13.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2284.1</td>
<td>2440.8</td>
<td>1254.3</td>
<td>13.20</td>
<td></td>
</tr>
</tbody>
</table>

The porosity increases with the increase in the rate of recycled aggregates. This is justified by the presence of two layers of cement: the new cement paste created by the water + cement mixture as well as another layer of old cement enveloping natural aggregates. The determination of the conformability of concrete from a durability standpoint depends on several factors essentially the probable life of the project, which depends on the type of the project, and external conditions of the project. Based on the performance approach and within the framework of the thesis project which consists of using concrete in an ordinary building in an ordinary environment, the concrete complies with conditions of the approach for all constructions with a lifespan of 100 years.

5. Conclusion and Future Work

This article summarizes all the tests carried out as part of a thesis on the characterization of recycled aggregates. These tests can be divided into 2 parts. The first part of tests is related to the mechanical characterization based on compression strength tests, the main conclusion is that the compressive strength depends on the percentage of replacement. It decreases with the increase in the percentage of recycled aggregates. Beyond 25% replacement, the resistance drops considerably and the use of recycled aggregate concrete is not recommended in structural elements. The ad of plasticizer improves strength values and the replacement level might increase to 50% when replacing 1% of cement by plasticizer.

The second part focuses on testing the porosity of RAC as indicator of its durability according to the performance approach. The test concludes that for a building with a required life of 100 years or less located in an ordinary environment: Dry, Wet or exposed to sea salts but without being in contact with it, our RAC is conform to the durability criteria.

The next step of research should focalize on an economic study of replacing naturel aggregates by recycled ones depending on several conditions (distance between the old building, the crushing area and the project, the quantity of concrete produced by recycled aggregates and the quantity needed for the project, etc.).

6. Declarations

6.1. Author Contributions

Conceptualization, K.N.; methodology, K.N.; validation, T.C.; writing—original draft preparation, K.N.; writing—review and editing, K.N.; supervision, K.N. 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


Limitations on ACI Code Minimum Thickness Requirements for Flat Slab

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Abstract

Reinforced concrete two-way flat slabs are considered one of the most used systems in the construction of commercial buildings due to the ease of construction and suitability for electrical and mechanical paths. Long-term deflection is an essential parameter in controlling the behavior of this slab system, especially with long spans. Therefore, this study is devoted to investigating the validation of the ACI 318-19 Code long-term deflection limitations of a wide range of span lengths of two-way flat slabs with and without drop panels. The first part of the study includes nonlinear finite element analysis of 63 flat slabs without drops and 63 flat slabs with drops using the SAFE commercial software. The investigated parameters consist of the span length (4, 5, 6, 7, 8, 9, and 10 m), compressive strength of concrete (21, 35, and 49 MPa), the magnitude of live load (1.5, 3, and 4.5 kN/m²), and the drop thickness (0.25t_slab, 0.5t_slab, and 0.75t_slab). In addition, the maximum crack width at the top and bottom are determined and compared with the limitations of the ACI 224R-08. The second part of this research proposes modifications to the minimum slab thickness that satisfy the permissible deflection. It was found, for flat slabs without drops, the increase in concrete compressive strength from 21MPa to 49MPa decreases the average long-term deflection by (56, 53, 50, 44, 39, 33 and 31%) for spans (4, 5, 6, 7, 8, 9, and 10 m) respectively. In flat slab with drop panel, it was found that varying drop panel thickness t from 0.25t_slab to 0.75t_slab decreases the average long-term deflection by (45, 41, 39, 35, 31, 28 and 25%) for span lengths (4, 5, 6, 7, 8, 9 and 10 m) respectively. Limitations of the minimum thickness of flat slab were proposed to vary from Ln/30 to Ln/19.9 for a flat slab without a drop panel and from Ln/33 to Ln/21.2 for a flat slab with drop panel. These limitations demonstrated high consistency with the results of Scanlon and Lee's unified equation for determining the minimum thickness of slab with and without drop panels.

Keywords: Long-term Deflection; Allowable Deflection; Flat Slab; Drop Panel Thickness; Concrete Compressive Strength; Crack width; Span Length.

1. Introduction

A flat plate slab (or known also as a flat slab without a drop panel) is a two-way reinforced concrete slab that transfers loads directly to the supporting columns without the aid of beams or drop panels or capitals. In case of the presence of column capitals, drop panels, or both the slab is called a flat slab. The flat plate, that is common in

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residential building, has several advantages such as cost savings due to low story height and simple/quick construction and formwork, and flat ceiling that has high fire resistance (few sharps corners for concrete spalling) and less obstruction to light diffusion. The flat slab is satisfactory for long spans and heavy loads, in particular, the flat slab is economical for parking, warehouses, and industrial buildings [1-3].

The deflection is a crucial issue in the design of flat slabs with or without drop panels. Most Standards like ACI 318-19 [4], CSA A23.3-04 [5], AS 3600 [6] and Euro code 2 [7] propose two alternative ways for the control of deflection. The first approach is to calculate the deflection and to compare the calculated deflection with the allowable limits. The second approach controls indirectly the deflection by limiting minimum slab thickness or maximum span/depth ratio.

The flexural stiffness EI (E: the concrete modulus of elasticity and I: the moment of inertia) of a flexural member is an essential variable in the calculation of deflection. For the reinforced concrete members, the amount of section cracking affects significantly the moment of inertia and consequently, this effect must be considered in the analysis of deflection [8]. Generally, there are two different methods for considering the cracking effect: the effective moment of inertia method [9] and the mean curvature method [10]. Furthermore, creep and shrinkage have important effects on the long-term deflection, and therefore literature provides several ways for considering this effect, the most famous one is the ACI 318 method. The analysis for deflection can be done by using a range of refined methods [11], like a non-linear analysis or finite element analysis [12]. Recent work has used the Artificial Neural Network approach [13] for the prediction of deflection. However, the approaches for calculating the deflection in flat slabs are complicated and involving several approximations due to complex behavior at the service load stage (cracking, time-dependent effect, tension stiffening). Therefore, the direct calculation of deflection for the typical situations is impractical and engineers prefer to control the deflection using the minimum slab thickness or maximum span/depth ratio approach.

The minimum slab thickness or maximum span/depth ratio approach is the focus of many researches for decades. Several studies [14-18] have proposed different expressions for the maximum allowable span/depth ratio for slabs (including flat plate and flat slabs) considering the effects of different factors such as sustained load, aspect ratio, reinforcement ratio, support condition, concrete modulus of elasticity, target maximum permissible incremental deflection and long-term deflection effects.

Vollum and Hossain [19] have studied the span/depth rules given in Euro code 2 and they have found that the deflections calculated in flat slabs dimensioned with span/depth rules of Euro code 2 can be excessive in external and corner panels since the rules fail to allow for the effect of cracking during construction. Lee and Scanlon [20] have compared the minimum slab (one-way and two-way) provisions of various Standards (ACI 318-08, Euro code 2, BS 8110-1:1997, and AS 3600-2001 and the unified equation proposed by Scanlon and Lee [15]) by performing a parametric study to evaluate the effects of several relevant design parameters. The results show that ACI 318 conditions need a revision to cover the range of the affected design parameters. Furthermore, applicability limitations require to be added to ACI provisions, especially for flat slab provisions which seem to be sufficient for the limit of L/240 (for typical loading and spans) but insufficient in many cases for the limit of L/480. Bertoro [21] has investigated the effectiveness of ACI 318 provisions for minimum thickness of two-way slabs for controlling the deflection to be within the allowable limits. This study evaluates (from a statistical viewpoint) the calculated deflections for two-way slabs having minimum thickness specified according to the ACI 318-14 requirements and as a result, it provides recommendations for upcoming ACI code revision. Hasan and Taha [22] have investigated the effects of several parameters (aspect ratio, live load, concrete strength) on the long-term deflection of flat plate slabs without edge beams (corner panels). They have highlighted the effect of not account for the aspect ratio in five Standards (ACI 318-14, CSA A23.3-14, AS 3600, BS8110, Euro code 2) provisions for the minimum slab thickness. Moreover, the applicability of the ACI 318-14 requirements for the thickness of flat plate slab without edge beam appeared to be sufficient to satisfy the permissible deflection limits L/360 and L/240 for typical spans and concrete strength while they were insufficient in many cases for the limit of L/480. Sanabra-Loewe et al. [23] have assessed the ACI 318 code and Eurocode 2 methods for the minimum slenderness ratio of R.C. slabs. The evaluated factors were: load, span, and permissible deflection. The results highlight the shortcoming of the Eurocode 2 and ACI 318 code provisions. Al-Nu’man & Abdullah [24] have developed a simulation model that considering the materials and loads uncertainties and along with the sensitivity analysis of results. The results indicate that the ACI 318-14 minimum thickness requirements are adequate for 4m and 6m span or less for flat plate and flat slab respectively. Depending on the characteristic strength of concrete, the redistribution factor, and the total steel ratio, Santos and Henriques [25] have proposed new span/depth limits satisfying both deflection and ductility requirements. However, these limits are restricted to the cases of beams and one-way slabs.

From the above review of literature, it is clear that there is a common consensus that the minimum thickness provisions required by ACI 318 code for flat slabs cannot ensure the deflection to comply with the maximum permissible limits for all flat slabs. Therefore, the objective of the present paper is to study the domain of applicability of ACI minimum thickness provisions for flat slab for controlling the long-term deflection and to provide the
community of engineers the limitations for these provisions. The present paper addresses this issue by selecting the slab thickness according to the ACI 318-19 provisions, then, calculating the deflections using the Nonlinear Finite element Analysis for 126 case studies of flat slabs (with and without drop panels) for a range of span lengths and practical selected values of several influencing parameters (live loads, materials strengths, and drop panel thickness) and comparing the computed deflections with the ACI 318-19 permissible values (L/240, L/480).

2. Nonlinear Finite Element Analysis

The methodology of the present study is devoted to calculate the long-term deflection of flat slabs with thicknesses that determined according to ACI 318-19 Code minimum thickness requirements and to compare the calculated deflections with ACI 318-19 Code permissible limits. To achieve this goal, a nonlinear Finite Element Analysis was performed to investigate the long-term deflection in flat slabs. The SAFE software was considered here for this purpose. The long-term deflection was calculated according to the procedure illustrated in [26]. This procedure includes the calculation of deflection for three cases:

- Case 1: the immediate deflection due to short-term loads: DL + SDL + LL,
- Case 2: the immediate deflection due to sustained loads: DL + SDL + ΨL LL,
- Case 3: the long-term deflection due to sustained loads: DL + SDL + ΨL LL.

Where DL, SDL and LL represent the slab self-weight, superimposed dead load and live load applied on the slab respectively. ΨL is the percentage of live load considered to be sustained.

Using SAFE software analysis options, the nonlinear (cracked) analysis was performed for cases 1 and 2, instead, for case 3 the nonlinear (long-term cracked i.e. with creep and shrinkage effects) analysis was carried out.

The value of long-term deflection was determined as a linear combination of case 3 + case 1 - case 2, where the difference between case 1 and case 2 represents the incremental deflection (without creep and shrinkage) due to non-sustained loading on a cracked structure.

Two layouts of the flat slabs were considered for analysis in the present study. both cases consist of three equal spans in each direction without edge beams, however, the first one is without drop panels (i.e. flat plate), see Figure 1, and the second layout with drop panels as shown in Figure 2. The drop panel dimensions were selected to comply with ACI 318-19 requirements as detailed in Figure 3.

The ACI code provisions for the minimum thickness of flat slab take into account only two effects: span length and yield strength of steel fy. However, this paper considers the effects of several factors on the long-term deflection and as a result on the minimum thickness requirements, these are: span length L, concrete compressive strength fc', service live load, and drop panel thickness t2. The range of values for each one of these factors was selected to be consistent with that used in the real practice and with available ACI 318-19 provisions. The selected values were: span length L (4, 5, 6, 7, 8, 9, 10) m, concrete compressive strength fc' (21, 35, 49) MPa, service live load (1.5, 3, 4.5) kN/m2, and drop panel thickness t2 (0.25t slab, 0.5t slab, 0.75t slab). On the contrary, the other parameters were considered fixed through the analysis and their specified values were:

- Steel reinforcement properties: yield strength fy = 420 MPa (Grade 60), Modulus of elasticity Es = 200 GPa,
- Modulus of elasticity of concrete Ec = 4700√fc',
- Superimposed dead load = 2 kN/m2,
- Dimension of squared columns supporting flat slabs with span length 4, 5, 6, 7, 8, 9 and 10 m are 300, 300, 350, 400, 450, 500 and 550 mm respectively,
- The percentage of live load that considered to be sustained ΨL = 25%.
- The time-dependent factor or creep coefficient = 2, i.e. for sustained load duration five years or more as specified in ACI 318-19.
Consequently, in total 126 case studies of flat slabs were analyzed to study the effects of factors considered in this paper. These case studies were divided equally into two main groups. The first one includes 63 case studies of the flat slab without drop panels and the second one comprises 63 case studies of flat slabs with drop panels. The two groups were similar in the range of values for span length and live load (values stated above), however, the concrete compressive strength was varied in the first one and had a fixed value $f'_c = 21$ MPa in second group. Furthermore, the range of values for drop panel thickness (given above) was considered in the second group only.

3. Results and Discussion

Using the nonlinear Finite Element Analysis, the long-term deflection was investigated at different points of 126 case studies of flat slabs. Figures 4 and 5 show the resulting long-term deflection for two extreme case studies of the flat slab without drop panel having the same concrete strength ($f'_c = 21$MPa) and live load (LL=4.5 kN/m$^2$) but with different values for span length (L=4m for Figure 4 and L=10m for Figure 5). Figures 6 and 7 illustrate the long-term deflection for another two case studies similar to that shown in Figures 4 and 5 respectively but for the flat slab with a drop ($t_d = 0.25t_{slab}$). From these four figures, it is clear that the maximum long-term deflection occurs at corner panels and nearly at the midpoint of the diagonal line between the corner and interior columns. The same finding was drawn from all other cases and therefore the long-term deflection given in the next sections will be at the midpoint of the diagonal line between the corner and interior columns for the corner panels.

Due to the large campaign of case studies considered in the present paper, it is convenient to discuss the results into two subsections, firstly for the cases of flat slabs without drop panel and secondly for the cases of flat slabs with drop panel.
3.1. Two-way Flat Slab without Drop Panels

Analysis results of maximum long-term deflection for the 63 case studies of the flat slab without drop panel are given in Table 1 and shown graphically in Figures 8, 9 and 10. As shown, the results were obtained from analyzing flat slabs having span lengths varied from 4 to 10 m, and for three values of concrete compressive strengths (21, 35, 49) MPa and three values of live loads (1.5, 3, 4.5) kN/m². The resulting maximum long-term deflections were compared with the ACI 318-19 allowable deflection limits: L/480 (roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections) and L/240 (roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections). Although the slab thickness was dimensioned according to ACI 318-19 minimum thickness requirements (L/30) for all cases, the calculated maximum long-term deflection exceeds one or both allowable limits in many cases. As an example, for the cases with LL=1.5 kN/m², the calculated deflections exceed the limit of L/240 when the span length is larger than 4, 6, 8 m for \( f'_c \) values of 21, 35, 49 MPa respectively. Furthermore, Figures 8, 9 and 10 show a nearly linear increase in maximum long-term deflection as the span length changes from 4 to 10 m, but with a slope that becomes steeper for weak concrete strength. In other words, improving the concrete compressive strength from 21 to 49 MPa reduces the maximum long-term deflection by an average of (56, 53, 50, 44, 39, 33 and 31%) for spans (4, 5, 6, 7, 8, 9 and 10 m) respectively. These percentages indicate that the efficiency of using stronger concrete (\( f'_c = 49 \) MPa) is the highest when the slab span length is 4 m. Regarding the effect of live loads, as expected, changing the live load from 1.5 to 4.5
kN/m² leads to more deflection, however, this effect is more pronounced for a small span length of 4 m and is diminished gradually for a larger span length. This behavior can be explained by referring to any short-term deflection elastic equation (for example wL³/384EI) where the span length L has power 4 while the loads w has power 1 and consequently the effect of the increase in span length is dominated.

Table 1 also compares the maximum cracks width at the top and bottom faces of the slab with the ACI 224R-08 [27] allowable limit of 0.3 mm that corresponds to the exposure condition: humidity, moist air and soil. From these analysis results, there is a clear trend of increasing the crack width with the increase in span length and as a result exceeding the allowable limits 0.3 mm for span length more than 7 m.

Table 1. Analysis results for flat slab without drop panel with different values of spans length, concrete compressive strength and live loads

<table>
<thead>
<tr>
<th>Span (L), m</th>
<th>t_slab</th>
<th>f’c = 21 MPa</th>
<th>f’c = 35 MPa</th>
<th>f’c = 49 MPa</th>
<th>Allowable</th>
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<td></td>
<td></td>
<td>long term def</td>
<td>maximum crack width</td>
<td>long term def</td>
<td>maximum crack width</td>
<td>long term def</td>
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<tr>
<td>L / 30 mm</td>
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<td>(mm)</td>
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<td>(mm)</td>
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<tr>
<td>4</td>
<td>125</td>
<td>12.6</td>
<td>0.12 0.15</td>
<td>8.1</td>
<td>0.17 0.12</td>
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<tr>
<td>5</td>
<td>160</td>
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<td>0.18 0.17</td>
<td>12.7</td>
<td>0.19 0.17</td>
<td>10.5</td>
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<tr>
<td>6</td>
<td>190</td>
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</tr>
<tr>
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<td>0.32 0.38</td>
<td>41.2</td>
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LL = 3 kN/m²

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<td>L / 30 mm</td>
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<td>maximum crack width</td>
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<td>maximum crack width</td>
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<td>(mm)</td>
<td>(mm)</td>
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<td>190</td>
<td>40.4</td>
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LL = 4.5 kN/m²

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<tr>
<td>L / 30 mm</td>
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<td>maximum crack width</td>
<td>long term def</td>
<td>maximum crack width</td>
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<tr>
<td>4</td>
<td>125</td>
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<td>190</td>
<td>44.9</td>
<td>0.21 0.22</td>
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<tr>
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<td>74.3</td>
<td>0.35 0.35</td>
<td>60.7</td>
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3.2. Two-way Flat Slab with Drop Panels

From the analysis of 63 case studies of the flat slab with drop panel, Table 2 provides the resulting maximum long-term deflections, also, Figures 11, 12, and 13 show these results graphically. The variables in this analysis were the span lengths (varied from 4 to 10 m), live loads (1.5, 3, 4.5) kN/m² and drop panel thickness $t_2$ (0.25$t_{slab}$, 0.5$t_{slab}$, 0.75$t_{slab}$). Since the effect of concrete compressive strength became clear from the above analysis of the flat slabs without drop panel, a fixed value of $f'_c=21$MPa was considered here for the analysis of flat slabs with drop panel. The resulting maximum long-term deflections were compared with the ACI 318-19 allowable deflection limits $L/480$ and $L/240$. In spite of the slab thickness was selected to comply with ACI 318-19 minimum thickness requirements ($Ln/33$) for all cases, the computed maximum long-term deflection exceeds one or both allowable limits in many cases. As an example, for the cases with LL=1.5 kN/m², the calculated deflections exceed the limit of L/240 when the span length is larger than 4, 5, 7 m for drop panel thickness $t_2$ of 0.25$t_{slab}$, 0.5$t_{slab}$, 0.75$t_{slab}$ respectively. Moreover, Figures 11, 12 and 13 show a nearly linear relation between the resulting maximum long-term deflection and the span length. However, the slopes of these relations reduce as the drop panel becomes thicker. In other words, varying drop panel thickness $t_2$ from 0.25$t_{slab}$ to 0.75$t_{slab}$ decreases the average long-term deflection by (45, 41, 39, 35, 31, 28 and
25%) for span lengths (4, 5, 6, 7, 8, 9 and 10 m) respectively. These percentages show that the positive effect of drop panel thickness is important for small spans and it becomes less significant for larger spans. Concerning the live load effect, a similar finding to that drawn above for flat slab without drop was found here i.e. increasing the live load leads to larger long-term deflection but this effect becomes less important with the increase in span lengths.

In addition to the maximum long-term deflection, Table 2 shows the resulting maximum cracks width at the top and bottom faces of slab. These results exhibit a logical increase in the width of the cracks as the span length varies from 4 to 10 m. The comparison of the resulting maximum cracks width with the ACI 224R-08 [27] allowable limit of 0.3 mm (that corresponds to the exposure condition: humidity, moist air and soil) indicates that the crack width fails to comply with the allowable limit (0.3 mm) when the span length is more than 7 m.

### Table 2. Analysis results for flat slab with drop panel with different values of spans length, drop panel thickness and live loads

<table>
<thead>
<tr>
<th>Span (L) m</th>
<th>t_{slab}</th>
<th>t_{2=0.25t_{slab}}</th>
<th>t_{2=0.5t_{slab}}</th>
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<th>Allowable deflections (mm)</th>
<th>Allowable crack width mm</th>
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<th>Allowable crack width mm</th>
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<td>84.5</td>
<td>0.30</td>
<td>0.35</td>
<td>145</td>
<td>75.1</td>
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</table>
4. Proposed Minimum Thickness of Flat Slab

4.1. Modifications of the ACI-318 Code Limitations

Based on the above discussion of the results obtained in the present study, it is clear that for the control of deflection the use of a single formula for the minimum thickness for all flat slabs without a drop (Ln/30) or with drop (Ln/33) as specified by ACI code (for $f_y = 420$ MPa, exterior panel without edge beam) is a serious issue. The main shortcoming of the ACI formulas is its restriction to a single variable (span length) and the ignoring of other influencing factors like concrete compressive strength, applied live loads, and drop panel thickness.

Consequently, the 126 case studies considered here were re-analyzed using the nonlinear finite element analysis in order to specify, for each case, the appropriate minimum thickness that can ensure the complying of long-term deflection with the allowable limit of L/240. For this purpose, the re-analysis was performed with a gradual increase in the slab thickness (increments of 5 mm) for each case and then the maximum long-term deflection was investigated and compared the limit L/240. According to ACI 318-19 code, in any case, the flat slab thickness should be at least 125 mm for slab without a drop and 100 mm for slab with a drop, therefore these values were considered as the starting values for the slab thickness in the analysis.

Table 3 gives the analysis results for the 63 cases of the flat slab without a drop. It shows, for each case study, the resulting appropriate minimum slab thickness and the corresponding maximum calculated long-term deflection. Based
on these results, a new proposed formula for minimum slab thickness that corresponds to each case study was proposed and presented in Table 3. As shown, these formulas vary from $L_n/30$ to $L_n/19.9$ which is a wide range as compared with the single formula provided by ACI code ($L_n/33$).

Regarding the re-analysis of the 63 cases of the flat slab with a drop, the analysis results were given in Table 4. These results include, for each case study, the investigated appropriate minimum slab thickness, the maximum computed long-term deflection and as a result the proposed new formula for the minimum slab thickness. As shown, the proposed formulas for the cases of slab with drop panel have a range from $L_n/33$ to $L_n/21.2$ which provides evidence that the single ACI code formula ($L_n/33$) cannot be satisfactory for all cases.

Table 3. Proposed minimum thickness of flat slab without drop panels that satisfies the ACI limit L/240

<table>
<thead>
<tr>
<th>Span (L.)</th>
<th>$f'_e = 21$ MPa</th>
<th>$f'_e = 35$ MPa</th>
<th>$f'_e = 49$ MPa</th>
<th>Allowable deflections (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>$L_n$ A</td>
<td>$t_{slab}$ mm</td>
<td>$L_n$ A</td>
<td>$t_{slab}$ mm</td>
</tr>
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<td>15.1</td>
<td>$L_n$ 30.0</td>
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<tr>
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<td>180</td>
<td>20.2</td>
<td>$L_n$ 30.0</td>
</tr>
<tr>
<td>6</td>
<td>$L_n$ 25.5</td>
<td>220</td>
<td>25.0</td>
<td>$L_n$ 28.2</td>
</tr>
<tr>
<td>7</td>
<td>$L_n$ 27.5</td>
<td>280</td>
<td>27.5</td>
<td>$L_n$ 26.9</td>
</tr>
<tr>
<td>8</td>
<td>$L_n$ 21.7</td>
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<td>32.0</td>
<td>$L_n$ 26.4</td>
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<td>9</td>
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<th>Span (L.)</th>
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<th>$f'_e = 49$ MPa</th>
<th>Allowable deflections (mm)</th>
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</thead>
<tbody>
<tr>
<td>m</td>
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<td>$L_n$ 25.5</td>
<td>235</td>
<td>25.0</td>
<td>$L_n$ 26.9</td>
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<tr>
<td>7</td>
<td>$L_n$ 27.5</td>
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<td>28.8</td>
<td>$L_n$ 25.8</td>
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<tr>
<td>8</td>
<td>$L_n$ 21.7</td>
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<td>33.3</td>
<td>$L_n$ 24.7</td>
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<tr>
<td>9</td>
<td>$L_n$ 20.5</td>
<td>405</td>
<td>36.7</td>
<td>$L_n$ 23.9</td>
</tr>
<tr>
<td>10</td>
<td>$L_n$ 19.9</td>
<td>475</td>
<td>41.6</td>
<td>$L_n$ 23.0</td>
</tr>
</tbody>
</table>
Table 4. Proposed minimum thickness of flat slab with drop panels that satisfies the ACI limit L/240

| Span (L) m | L.L=1.5 kN/m² | | | | L.L=3 kN/m² | | | | L.L=4.5 kN/m² | |
|-----------|----------------|-----------------|-----------------|----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
|           | t₂=0.25L₀| t₂=0.5L₀| t₂=0.75L₀| allowable | t₂=0.25L₀| t₂=0.5L₀| t₂=0.75L₀| allowable | t₂=0.25L₀| t₂=0.5L₀| t₂=0.75L₀| allowable |
|           | t₀ | t₁ | long term def (mm) | t₀ | t₁ | long term def (mm) | t₀ | t₁ | long term def (mm) | t₀ | t₁ | long term def (mm) | t₀ | t₁ | long term def (mm) |
| 4         | 115 | 29  | 12.8 | 115 | 58  | 9.1  | 115 | 87  | 7.2  | 115 | 29  | 12.8 | 115 | 58  | 9.1  | 115 | 87  | 7.2  | 16.6  |
| 5         | 150 | 38  | 21.7 | 145 | 73  | 18.8 | 145 | 109 | 13.1 | 16.6 |
| 6         | 200 | 50  | 25.0 | 185 | 93  | 24.6 | 175 | 132 | 20.5 | 25.0  |
| 7         | 235 | 59  | 29.0 | 220 | 110 | 29.1 | 200 | 150 | 26.7 | 29.1 |
| 8         | 290 | 73  | 33.3 | 260 | 135 | 31.5 | 240 | 180 | 32.0 | 33.3 |
| 9         | 340 | 85  | 37.5 | 305 | 153 | 37.5 | 280 | 210 | 36.5 | 37.5 |
| 10        | 405 | 102 | 41.1 | 370 | 185 | 41.0 | 340 | 255 | 40.0 | 41.6 |

4.2. Scanlon & Lee Unified Slab Thickness Equation

In 2006, Scanlon & Lee [15] presented a unified equation to estimate the minimum thickness for non-prestressed one-way and two-way slabs and beams. The proposed equation takes into account various parameters relating to the geometrical and material characteristics of the flat slab, such as the support conditions, the existence of drop panel, aspect ratio for edge supported slab, and the modulus of elasticity of concrete. The general form of the proposed equation is as follows:

\[
l_{\text{min}}/h = \beta \left( \frac{D_{\text{inc}}}{l_{\text{allow}}} \right) \left( \frac{0.0167 \times K_{SP} \times E_{C} \times b}{K \times K_{SP} \times K_{AR} \times (W_{S} + W_{L, \text{add}})} \right)^{1/3}
\]

(1)
where: \( L_h \): is the clear span in mm; \( h \): is the minimum thickness in mm; \( \beta \): edge support coefficient (for slab without edge support equals to 1.0 and for edge supported equals to long span / short span); \((\Delta_{\text{inc}}/L)_{\text{allow}}\): is the targeted incremental deflection which equals to 1/480 for flat slab; \( K_{DP} \): drop panel coefficient which equals to 1.0 for slabs without drop and 1.35 for slabs with drop panels; \( E_C \): is the modulus of elasticity of concrete; \( b \): is the strip width which equals to 1000mm; \( K \): is the coefficient of end support condition which equals to 1.4 for both ends continuous, 2.0 for one end continuous and 5.0 for both ends continuous; \( K_{SS} \): is the coefficient column supported condition of two way slabs which equal to 1.35 for column supported and 1.0 for other cases; \( K_{AB} \): is the edge support condition which equals to 0.2 + 0.4 \( \beta \) for edge supported slabs and 1.0 for other cases; \( \lambda \): is the time-dependent factor of sustained loads according to ACI 318-14 Code. \( W_1 \): is the sustained load in kN/m which equals to the self-weight plus superimposed dead load plus 0.25 of the live load; and \( W_{L_{\text{add}}} \): is the additional live load in kN/m² which equals to 0.75 of the live load.

For comparison reasons, Equation 1 is implemented on the investigated cases of slabs with and without drop panels. The results were listed in Tables 5 and 6. In general, high consistence was found between the results of the Scanlon and Lee equation and the proposed limitation especially for slabs without drop panels. Higher thickness was recorded by using the equations of Scanlon and Lee than the proposed limitations and the ACI-318 Code limitations. All the output of the equation and the proposed limitations were satisfied the required allowable deflection that indicated by the ACI-318 Code. That demonstrated the efficiency of the proposed limitations by means of agree with the results of the equation and at the same time satisfying the allowable deflection requirements. Moreover, the proposed limitations considered effect of thickness of the drop panels which is neglected in the Scanlon and Lee equation.

Table 5. Minimum thickness of flat slab without drop panels based on the proposed limitations, ACI-318 Code limitations and Scanlon and Lee equation

<table>
<thead>
<tr>
<th>Span (L) (L) m</th>
<th>( f'_{c} = 21\text{MPa} )</th>
<th>( f'_{c} = 35\text{MPa} )</th>
<th>( f'_{c} = 49\text{MPa} )</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Proposed t mm</td>
<td>ACI-318 t mm</td>
<td>Scanlon and Lee eq. t mm</td>
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<td>125</td>
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<td>222.0</td>
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<th>( f'_{c} = 35\text{MPa} )</th>
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Table 6. Minimum thickness of flat slab without drop panels based on the proposed limitations, ACI-318 Code limitations and Scanlon and Lee equation

Two-way flat slab with drop panels, $f_y = 420$ MPa, $f'_c = 21$MPa

<table>
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<td>Scanlon and Lee t mm</td>
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<td>Scanlon and Lee t mm</td>
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5. Conclusions

The nonlinear Finite Element Analysis was used in order to study the effectiveness of using ACI minimum thickness provisions for flat slab for the controlling of long-term deflection. The analysis involved 126 case studies that considered the effects of several influencing parameters: slab span length, concrete compressive strength, the applied live load, and the thickness of the drop panel. From the analysis results, the main findings can be summarized as follow:

- ACI 318-19 minimum thickness provisions required for the control of deflection in flat slab (with or without drop) cannot be satisfactory (i.e. to comply with the ACI allowable limits L/480 and L/240) for all cases because they consider the effects of only the span length and yield strength of steel and ignoring the effects of the other influencing factors like the concrete compressive strength, live load, and the drop panel thickness. Therefore, these ACI code provisions have a serious problem and need a real revision;

- The effect of using high concrete compressive strength on reducing the long-term deflection was found to be significant especially for small spans. It was observed that the increase in concrete compressive strength from 21MPa to 49MPa decreases the average long-term deflection by (56%, 53%, 50%, 44%, 39%, 33% and 31%) for spans (4, 5, 6, 7, 8, 9 and 10 m) respectively;
In flat slab with drop panel, the use of thicker drop panel has an important positive effect on the reduction of long-term deflection especially for small spans. It was found that varying drop panel thickness \( t_2 \) from 0.25\( t_{slab} \) to 0.75\( t_{slab} \) decreases the average long-term deflection by (45, 41, 39, 35, 31, 28 and 25%) for span lengths (4, 5, 6, 7, 8, 9 and 10 m) respectively;

Concerning the live load effect, it was observed that increasing the live load leads to larger long-term deflection but this effect becomes less important with the increase in span lengths;

Formulas for calculating the minimum thickness of flat slab were proposed to vary from \( L_t/30 \) to \( L_t/19.9 \) for flat slab without drop panel and from \( L_t/33 \) to \( L_t/21.2 \) for flat slab with drop panel;

High constancy was observed between the results of Scanlon and Lee equation and the proposed limitations of the minimum thickness of slabs with and without drop panels.

6. Declarations

6.1. Author Contributions


6.2. Data Availability Statement

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

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


Fuzzy Analytical Hierarchy Processes for Damage State Assessment of Arch Masonry Bridge

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Abstract

The present work proposes a fuzzy analytical hierarchy approach for decision making in the maintenance programming of masonry arch bridges. As a practical case, we propose to classify the degradation state of the Mohammadia masonry bridge. A large number of criteria and sub-criteria are combined to classify this type of bridges through visual inspections. The main criteria (level 1) considered in this work are the history of the bridge, the environmental conditions, the structural capacity and the professional involvement of the bridge. In addition, these criteria are subdivided into several sub-criteria (level 2) which are, in turn, subdivided into sub-criteria (level 3). Considering these criteria and sub-criteria, weights $W_i$ are calculated by fuzzy geometric mean method of Buckley. Subsequently, expert scores were assigned to calculate the overall score $CS$ reflecting the degradation of the considered infrastructure. Thereafter, the masonry arch bridges are classified respecting the French IQOA scoring system using the overall scores value $CS$. The proposed classification method gave similar results provided by an expert’s study realized previously as part of a national patrimony preservation policy. The obtained results are in good agreement, which makes this method an effective scientific tool for decision-making in view of prioritization of the maintenance after systematic inspection of masonry bridges such as the bridge studied in this work.

Keywords: Masonry Bridges; Fuzzy Analytical Hierarchy Process; Degradation Degree Score; Classification of Bridge Degradation.

1. Introduction

Masonry bridges constitute a significant heritage within the road network. The average life of these structures exceeds, on average, a century of service. Consequently, the management of this heritage is essential through rigorous monitoring and regular maintenance. This inspection task proved to be very complex, given the large number of factors and the complex nature of the decision-making problem [1]. In this context, early in 1980’s, Saaty [2] proposed an Analytical Hierarchy Method (AHP) to solve the problem of decision-making complexity. This latter used the hierarchical structure that helps experts to make a simple classification. Since this date, several efforts were made by researchers to perform and improve this method [3-7]. Indeed, the AHP method can be generalized to determine the risks associated with bridges [8-10]. Thus, a scoring system is combined to help engineers establish a bridge reinforcement scheme using the AHP approach [11-13].

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To overcome the problem of difficulty for experts to provide precise numerical values of scoring inherent to each criterion, scientists developed the Fuzzy Analytical Hierarchical Process (FAHP) [14–17]. This latter makes it possible for experts to give several indices for each factor. As an example, Sasmal and Ramanjaneyulu [15] proposed this methodology for grading the condition of reinforced concrete (RC) bridges and concluded that this helped engineers and decision-makers to overcome the problem of prioritizing bridge maintenance and rehabilitation. Moreover, the method was successfully applied for Risk prioritization in megaprojects [18]. In fact, these authors combined the fuzzy version with fuzzy TOPSIS to evaluate a risk ranking considering three criteria, namely, cost, time and quality of the project.

Furthermore, the FAHP method has been applied in the decision-making process for the purchase of a property in a competitive context where several choices are available. As such, Shahidan et al. [19] applied the FAHP method to help purchaser selecting car and comparing the important criteria and sub-criteria needed to choose easily a car in Malaysia. The latter suggest the methodology as a guide to be implemented to other multiple criteria decision-making problems.

The objective of this work is to apply the FAHP method in the case of a masonry bridge. This approach consists of evaluating the current state of this type of structure in order to present a guide for the future maintenance and management. The main criteria used to assess the degree of degradation are the history, environmental conditions, structural capacity and professional involvement of the bridge. These criteria are divided into a set of sub-criteria, as the case requires. Thereafter, score is assigned by a group of experts to each criterion and each sub-criterion. The strong point of the method is to multiply these scores by weights calculated by the FAHP method. The sum of these products determines an overall CS score which defines the degree of degradation of the bridge. Using the French IQOA scoring system, the structure is finally classified according to the value obtained from CS. A lower CS score indicates a good condition of the bridge. On the other hand, a high CS score indicates a very pronounced degradation state. Therefore, bridges with a high CS are to be prioritized during maintenance campaigns.

2. Methodology for Degradation Assessment Procedure

This study aims to develop a numerical model based on the FAHP method to assess the condition of masonry arch bridges. Figure 1 shows the flowchart giving the five steps of the model. As indicated by the flowchart, the first step is to identify the degradation factors having a direct influence on the condition state of the bridge. Secondly, the hierarchical structure of the model is built. Once done, the FAHP method can be applied in order to calculate the overall weights of the factors and the sub-factors as will be detailed later. In the next step, an overall CS score index is calculated and used to classify the degree of degradation of the masonry bridge in the last step.

![Flowchart of procedure for estimating degradation of masonry arch bridges](image)

2.1. Identification of Degradation Factors

Factors that might have an impact on the degradation of the structure were selected after a process of a census and collection. This selection was based on expert judgment after inspections and visual surveys without recourse to in-situ measurements and tests [20]. According to the literature and the assessments of experts, thirty-five factors and sub-factors were retained in this study. Among other criteria, the history plays a major role in the condition of the bridge. Combined with the environmental conditions, the age of the masonry bridge has a negative impact on its state of health. Added to this, the structural capacity of the bridge such as the apparent disorder of the foundations, deflection and deformation of the superstructure and thus the condition of the equipment are also retained. The last criterion retained is the professional involvement which contains design involvement and supervision of the structure.
2.2. Development of the Hierarchical Structure

The constructed hierarchical structure contains four levels as shown in Figure 2. Level 0 consists in defining the problem. This consists in evaluating the degree of degradation of the masonry bridge. Level 1 includes the four main criteria indicating the state of degradation of the studied structure. Level 2 identifies the different sub-criteria included in each criterion. The last level groups the sub-criteria that result from the last five sub-criteria of level 2.

![Figure 2. The hierarchy structure of adopted assessment criteria](image)

2.3. Calculation of Weights by the Fuzzy Analytic Process (FAHP) Method

It is difficult for experts to provide precise numerical values from comparison ratios for several reasons. Indeed, the ambiguity and the complexity of information emitted by human during decision making often arises. Appropriately, the Fuzzy Analytical Hierarchy Process (FAHP) approach is used in this study to deal with uncertainty in such situations.
2.3.1. The Triangular Fuzzy Numbers (TFN)

Among the various forms of fuzzy number, the Triangular Fuzzy Number (TFN) is the most widely used in the literature. This triangular fuzzy number denoted \( \tilde{A} \) is a function of a triplet \((l, m, u)\), where \( l \), \( m \) and \( u \) are the lower, middle and upper bounds of TFN \([21, 22]\). TFN is intuitive, easy to use, computationally simple and useful for processing calculation in a fuzzy environment. Figure 3 presents the membership function of TFN \([23]\) that is expressed as follows:

\[
\mu_{\tilde{N}}(x) = \begin{cases} 
\frac{x-l}{m-l}, & l \leq x \leq m \\
\frac{u-x}{u-m}, & m \leq x \leq u \\
0, & \text{otherwise}
\end{cases}
\] (1)

Figure 3. The curve of triangular fuzzy number \([23]\)

Given any two TFNs \( \tilde{A}_1 = (l_1, m_1, u_1) \) and \( \tilde{A}_2 = (l_2, m_2, u_2) \), the main operation of triangular fuzzy numbers are \([24]\):

\[
\tilde{A}_1 \oplus \tilde{A}_2 = (l_1, m_1, u_1) \oplus (l_2, m_2, u_2) = (l_1 + l_2, m_1 + m_2, u_1 + u_2) \\
\tilde{A}_1 \otimes \tilde{A}_2 = (l_1, m_1, u_1) \otimes (l_2, m_2, u_2) = (l_1 \times l_2, m_1 \times m_2, u_1 \times u_2) \\
\tilde{A}_1(\,/)\tilde{A}_2 = (l_1, m_1, u_1) (\,/) (l_2, m_2, u_2) = (l_1/u_2, m_1/m_2, u_1/l_2) \\
\tilde{A}_1^{-1} = (l_1, m_1, u_1)^{-1} = (1/u_1, 1/m_1, 1/l_1)
\] (2)(3)(4)(5)

2.3.2. The fundamental scale used to compare two criteria

To choose an appropriate TFN, Experts are invited to make a comparison of the relative importance of two criteria at the same time. Figure 4 illustrates the scaling scheme of the appreciation procedure.

2.3.3. Construction of Comparison Matrices

The pair-wise comparison matrix \([\tilde{A}]\) is constructed by collection of pair-by-pair scores as follows \([25, 26]\):

Figure 4: The fundamental scale used to compare a pair of criteria
\[
[\tilde{A}] = (\tilde{a}_{ij})_{n \times n} = \begin{bmatrix}
(1,1,1) & (l_{12}, m_{12}, u_{12}) & \cdots & (l_{1n}, m_{1n}, u_{1n}) \\
(l_{21}, m_{21}, u_{21}) & (1,1,1) & \cdots & (l_{2n}, m_{2n}, u_{2n}) \\
\vdots & \vdots & \ddots & \vdots \\
(l_{n1}, m_{n1}, u_{n1}) & (l_{n2}, m_{n2}, u_{n2}) & \cdots & (1,1,1)
\end{bmatrix} \quad (6)
\]

Where, i and j vary from 1 to n (number of parameters) and \((\tilde{a}_{ij})\) indicates the expert’s preference of \(i^{th}\) criterion over \(j^{th}\) criterion. The lower triangular matrix \([\tilde{A}]\) is computed by the Equation 7:

\[
(\tilde{a}_{ij}) = (\tilde{a}_{ij})^{-1} = \left(\frac{1}{u_{ij}}, \frac{1}{m_{ij}}, \frac{1}{l_{ij}}\right), \text{ for } i < j
\]  

All the comparison matrices between the criteria and sub-criteria of the assumed hierarchical structure will be presented in the results section.

2.3.4. Fuzzy Geometric Mean Method

According to Buckley [27], the matrix \([\tilde{A}]\) is aggregated by fuzzy geometric mean \(\tilde{r}_i\) using the expression:

\[
\tilde{r}_i = \left(\prod_{j=1}^{n} \tilde{a}_{ij}\right)^{\frac{1}{n}}, i = 1,2, \ldots, n \quad (8)
\]

2.3.5. Fuzzy Weights

Fuzzy weights \(\tilde{w}_i\) are computed by multiplying each fuzzy geometric mean \(\tilde{r}_i\) by a vector summation as follows [25]:

\[
\tilde{w}_i = (lw_i, mw_i, uw_i) = \tilde{r}_i \otimes (\tilde{r}_2 \oplus \cdot \cdot \cdot \oplus \tilde{r}_n)^{-1}, i = 1,2, \ldots, n \quad (9)
\]

After that, the fuzzy weights \(\tilde{w}_i\) must be defuzzified by the method known as center of area (COA) as follows [28]:

\[
w_i = \frac{lw_i + mw_i + uw_i}{3}, i = 1,2, \ldots, n \quad (10)
\]

This step is followed by the normalization of the defuzzified weights following the expression:

\[
nw_i = \frac{w_i}{\sum_{i=1}^{n} w_i}, i = 1,2, \ldots, n \quad (11)
\]

2.3.6. Consistency Test

In order to assure the consistency of the judgment matrix, a defuzzification process was performed using method proposed by Kwong and Bai [28]. Each TFN in the pair-wise matrix is converted to crisp number \(c_{ij}\) as follows:

\[
c_{ij} = \frac{(l_{ij} + 4m_{ij} + u_{ij})}{6}, i, j = 1,2, \ldots, n \quad (12)
\]

Then, the crisp pair-wise comparison matrix \([C]\) is constructed as follows [29, 30]:

\[
[C] = (c_{ij})_{n \times n} = \begin{bmatrix}
1 & c_{12} & \cdots & c_{1n} \\
c_{21} & 1 & \cdots & c_{2n} \\
\vdots & \vdots & \ddots & \vdots \\
c_{n1} & c_{n2} & \cdots & 1
\end{bmatrix} \quad (13)
\]

Next, the consistency index, CI [30, 31], is calculated as:

\[
CI = \frac{\lambda \text{ max } - n}{(n - 1)} \quad (14)
\]

Where \(\lambda \text{ max}\) is the maximum eigenvalue of the matrix\([C]\). The consistency ratio CR is given by the expression:

\[
CR = \frac{CI}{RI} \quad (15)
\]

Where, CI is the consistency index and RI is the Random Index given in Table 1. According to Saaty [31], matrices with CR values ≤10% are accepted, otherwise CR values greater than 10% are rejected.

<table>
<thead>
<tr>
<th>n</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>RI</td>
<td>0.525</td>
<td>0.882</td>
<td>1.109</td>
<td>1.248</td>
<td>1.342</td>
<td>1.406</td>
<td>1.450</td>
<td>1.485</td>
</tr>
</tbody>
</table>
2.4. Evaluation of the Degradation Degree Score

In this part of the research, once the overall weights are determined by the FAHP method, each factor has an expert score denoted "ESC", ranging from 0 to 10. The relationship between ESC and the scale is shown in Table 2.

<table>
<thead>
<tr>
<th>ESC</th>
<th>State condition of scaling</th>
</tr>
</thead>
<tbody>
<tr>
<td>[0, 2]</td>
<td>no risk grade noticed</td>
</tr>
<tr>
<td>[2, 4]</td>
<td>low-grade risk</td>
</tr>
<tr>
<td>[4, 6]</td>
<td>moderate risk</td>
</tr>
<tr>
<td>[6, 8]</td>
<td>high risk</td>
</tr>
<tr>
<td>[8, 10]</td>
<td>super high risk</td>
</tr>
</tbody>
</table>

Table 2. Risk scaling status [32]

Next, the average score $ACS$ is calculated by the Equation 16:

$$ACS = \frac{\sum_{i=1}^{n} ESC_i}{n}$$  \hspace{1cm} (16)

Where $n$ is the number of experts. Finally, the comprehensive score CS (commonly called the overall score) for degradation of masonry bridges [32] is defined by Equation 17:

$$CS = \sum_{i=1}^{26} W_i \times ACS_i$$  \hspace{1cm} (17)

2.5. Classification of Masonry Bridge Degradation According to IQOA

The French scoring system IQOA [33] (quality assessment of engineering structures) is used in this research to classifying the degree of degradation of masonry bridge. Table 3 illustrates the principle of evaluating and classifying structures using the overall value of the CS scores. Class 1 corresponds to a CS score of 0 to 2 and designed a good apparent condition. While Class 3U (CS score from 8 to 10) represents extremely critical condition with serious structural failure.

<table>
<thead>
<tr>
<th>Class</th>
<th>CS</th>
<th>Apparent Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>[0, 2]</td>
<td>Good overall state</td>
</tr>
<tr>
<td>2</td>
<td>[2, 4]</td>
<td>Equipment failures or minor structure damage. Non urgent maintenance needed</td>
</tr>
<tr>
<td>2E</td>
<td>[4, 6]</td>
<td>Equipment failures or minor structure damage. Urgent maintenance needed</td>
</tr>
<tr>
<td>3</td>
<td>[6, 8]</td>
<td>Structure deterioration. Non urgent maintenance needed</td>
</tr>
<tr>
<td>3U</td>
<td>[8, 10]</td>
<td>Serious structure deterioration. Urgent maintenance needed</td>
</tr>
</tbody>
</table>

Table 3. IQOA [33] classification to evaluate the degradation of masonry bridges

3. Case Study: Masonry Bridge of Mohammadia

3.1. Localization and Description of the Bridge

A masonry bridge located at Mohammadia city was selected for our case study. The bridge in question is located in the north-west of Algeria (Figure 5). This bridge measures 110 meters long and 7.50 meters wide including ten stone arch spans, as shown in Figure 6.

Figure 5. Location of the studied bridge, a) Global location into Algeria map, b) Zoom to Mascara province and c) Capture image near the bridge of Mohammadia city
It should be noted that the bridge has been closed to traffic since the commissioning of the new bridge as a result of the deviation of the roadway. Despite this, a rehabilitation operation is desired and remains to be included in the framework of the preservation of the national heritage. The photos in Figures 7.a to 7.d highlight the types of damage, stated during our on-site inspection of the structure.

![Figure 6. Overview of the studied masonry bridge: a) Upstream side and B) Downstream side](image)

![Figure 7. Photos showing the damage state revealed on the bridge a) Roadway, b) Abutment on the upstream side, c) Arc number one, upstream side and d) Arcs number four and five on the downstream side](image)

The construction of the bridge dates back to the colonial period so its age will exceed a century of service, according to preliminary information. This criterion works against the state health of the bridge. In addition, as shown in Figures 7(b-c), there are open transverse cracks in the abutment wall. The same figures show a massive degradation of the joints between the stones over the entire surface of the return wall, as well as the presence of vegetation. Additionally, Figure 7(d), shows advanced pavement settlement caused by very heavy scour under the foundation. In addition, very pronounced breaks in the body of the vault were noted with detachment of the stones (Figure 7.d). As a first judgment, this structure will be classified as highly deteriorated and therefore requiring major rehabilitation work. In the following sections, we will analyze the resulted weights values performed using the fuzzy analytical hierarchy approaches. This approach will allow us to determine the overall score which determines the classification of the structure and to make a decision concerning the urgency to proceed to maintenance and rehabilitation of the bridge.

3.2. Results Analysis

The calculations at the base of the algorithm explained in section 2 were carried out using the MS Excel environment. Indeed, MS Excel remains the most widespread and the most accessible to the greatest number of designers and
researchers. Table 4 summarizes the triangular fuzzy numbers pair-by-pair of all criteria and sub-criteria. According to expert’s assessments, it is assessed, for example that criterion C1 is of equal importance with itself. This is interpreted by the fuzzy triangular number TFN (1, 1, 1) (see Figure 4). Similarly, criterion C1 is classified as low intermediately to C2, which is interpreted by a TFN (1/3, 1/2, 1). In the same time, this same criterion C1 is classified very weak intermediately to C3, which is interpreted by a TFN (1/7, 1/6, 1/5). On the other hand, C1 is classified moderately strong intermediately to C4. This is interpreted by a TFN (1, 2, 3). Thus, all the comparison matrices between the criteria and sub-criteria of the hierarchical structure are constructed as presented in Table 4.

Table 4. Pair-wise comparison matrices according to the assumed hierarchical structure

<table>
<thead>
<tr>
<th>Factors</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>(1,1,1)</td>
<td>(1/3,1/2,1)</td>
<td>(1/7,1/6,1/5)</td>
<td>(1,2,3)</td>
</tr>
<tr>
<td>C2</td>
<td>(1,2,3)</td>
<td>(1,1,1)</td>
<td>(1/4,1/3,1/2)</td>
<td>(2,3,4)</td>
</tr>
<tr>
<td>C3</td>
<td>(5,6,7)</td>
<td>(2,3,4)</td>
<td>(1,1,1)</td>
<td>(4,5,6)</td>
</tr>
<tr>
<td>C4</td>
<td>(1/3,1/2,1)</td>
<td>(1/4,1/3,1/2)</td>
<td>(1/6,1/5,1/4)</td>
<td>(1,1,1)</td>
</tr>
<tr>
<td>C1.1</td>
<td>(1,1,1)</td>
<td>(1/3,1/2,1)</td>
<td>(1/3,1/2,1)</td>
<td>(1,1,1)</td>
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<td>(1,2,3)</td>
<td>(1,1,1)</td>
<td>(1/3,1/2,1)</td>
<td>(1,2,3)</td>
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<tr>
<td>C1.3</td>
<td>(1,2,3)</td>
<td>(1,1,1)</td>
<td>(1,1,1)</td>
<td>(1,1,1)</td>
</tr>
<tr>
<td>C1.4</td>
<td>(1,1,1)</td>
<td>(1/2,3)</td>
<td>(1,1,1)</td>
<td>(1,1,1)</td>
</tr>
<tr>
<td>C1.5</td>
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<td>(1,1,1)</td>
</tr>
<tr>
<td>C2.1</td>
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<td>(1,1,1)</td>
<td>(1,1,1)</td>
<td>(1,2,3)</td>
</tr>
<tr>
<td>C2.2</td>
<td>(1,1,1)</td>
<td>(1,1,1)</td>
<td>(1/2,3)</td>
<td>(3,4,5)</td>
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<tr>
<td>C2.3</td>
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<td>(1,1,1)</td>
<td>(3,4,5)</td>
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<td>C2.5</td>
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<td>(1/4,1/3,1/2)</td>
<td>(1/5,1/4,1/3)</td>
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</tr>
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<td>(1,1,1)</td>
<td>(4,5,6)</td>
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<td>(5,6,7)</td>
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<td>(1/7,1/6,1/5)</td>
<td>(1,1,1)</td>
<td></td>
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<td>(1,1,1)</td>
<td>(1/3,1/2,1)</td>
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<td>(3,4,5)</td>
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<td>(2,3,4)</td>
</tr>
<tr>
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<td>(1/4,1/3,1/2)</td>
<td>(1,1,1)</td>
</tr>
<tr>
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<td>(1/2,3)</td>
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<tr>
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<tr>
<td>C4.1</td>
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<td>(1,1,1)</td>
<td></td>
</tr>
</tbody>
</table>
Table 5 summaries local weights $w_i$ obtained after realizing the consistency tests. These results obtained by the FAHP are projected on the hierarchical structure as illustrated by Figure 8.

<table>
<thead>
<tr>
<th>$w_i$</th>
<th>Consistency test</th>
<th>$\lambda_{max}$</th>
<th>$CI$</th>
<th>$RI$</th>
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<tr>
<td>0.125</td>
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<td>4.106</td>
<td>0.035</td>
<td>0.882</td>
<td>3.92% &lt; 10%</td>
</tr>
<tr>
<td>0.225</td>
<td></td>
<td>5.190</td>
<td>0.047</td>
<td>1.109</td>
<td>4.24% &lt; 10%</td>
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<tr>
<td>0.565</td>
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<td>5.273</td>
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<td>1.109</td>
<td>6.09% &lt; 10%</td>
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<tr>
<td>0.085</td>
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<td>0.002</td>
<td>0.580</td>
<td>0.525</td>
<td>0.41% &lt; 10%</td>
</tr>
<tr>
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<td>5.197</td>
<td>0.049</td>
<td>1.109</td>
<td>4.41% &lt; 10%</td>
</tr>
<tr>
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<td></td>
<td>0.356</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>0.221</td>
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<td>4.033</td>
<td>0.011</td>
<td>0.882</td>
<td>1.21% &lt; 10%</td>
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<td>0.471</td>
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<td>0.356</td>
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<td>0.471</td>
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<tr>
<td>0.085</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In fact, "structural capacity of the bridge" received a maximum importance (56.5%) followed by "environmental conditions" (22.5%), and in last position, "bridge history state" (12.5%) and "professional involvement conditions" (8.5%) received a relatively minimal effect importance in the degradation process of the masonry bridge. According to the hierarchical model, the overall weights $W_i$ of sub-factors are equal to the product of local weights of their father factors (of lower level). Table 6 summarizes $W_i$ results with the source local weights by levels. Visibly, analyzing Figure 9, one can easily note that there are considerable differences in the overall weights relating to the retained sub-factors. These differences relate their importance and influences on the state of degradation masonry bridge.
For a validation purpose, the obtained results are compared to the results published by Bakhtavar et al. [34]. In the latter, authors presented an example of weights calculation by the fuzzy geometric mean method. Observing Figure 10, one can easily note that the current results are in good agreement with those given by Bakhtavar et al.

![Figure 9. Overall weights of sub-factor](image)

![Figure 10. Results comparison of the calculated fuzzy weights to those obtained by [34]](image)

At this stage, three experts are invited to give notes (ESC1, ESC2 and ESC3) to assess the degradation state of the bridge. Indeed, for each factor, three scores were assigned as given in Table 7. The average of the scores to each sub-criterion is multiplied by the overall relating weight. The last calculation to be made in this case study is to calculate the overall score as expressed by Equation 17. The resulted overall score is given at the end of Table 7. Accordingly, the structure is finally classified according to the IQOA grading system as 3U, which is equivalent to a severely deteriorated state of the bridge. This means that immediate maintenance is necessary and imperative.
Table 7. The final results and the classification of the studied masonry bridge

<table>
<thead>
<tr>
<th>Number</th>
<th>Factor</th>
<th>ESC 1</th>
<th>ESC2</th>
<th>ESC3</th>
<th>Average score (ASC_i)</th>
<th>overall weight (W_i)</th>
<th>ASi x Wi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C1.1</td>
<td>8.000</td>
<td>9.000</td>
<td>9.000</td>
<td>8.667</td>
<td>0.020</td>
<td>0.170</td>
</tr>
<tr>
<td>2</td>
<td>C1.2</td>
<td>8.000</td>
<td>9.000</td>
<td>9.000</td>
<td>8.667</td>
<td>0.029</td>
<td>0.250</td>
</tr>
<tr>
<td>3</td>
<td>C1.3</td>
<td>8.000</td>
<td>7.000</td>
<td>6.000</td>
<td>7.000</td>
<td>0.027</td>
<td>0.191</td>
</tr>
<tr>
<td>4</td>
<td>C1.4</td>
<td>8.000</td>
<td>9.000</td>
<td>10.000</td>
<td>9.000</td>
<td>0.027</td>
<td>0.246</td>
</tr>
<tr>
<td>5</td>
<td>C1.5</td>
<td>8.000</td>
<td>10.000</td>
<td>10.000</td>
<td>9.333</td>
<td>0.022</td>
<td>0.201</td>
</tr>
<tr>
<td>6</td>
<td>C2.1</td>
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<td>7.000</td>
<td>8.000</td>
<td>7.667</td>
<td>0.050</td>
<td>0.382</td>
</tr>
<tr>
<td>7</td>
<td>C2.2</td>
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<td>0.072</td>
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<tr>
<td>8</td>
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<td>8.000</td>
<td>9.000</td>
<td>8.333</td>
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<td>0.500</td>
</tr>
<tr>
<td>9</td>
<td>C2.4</td>
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<td>4.000</td>
<td>6.000</td>
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<tr>
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<td>0.020</td>
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<td>6.333</td>
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<td>9.000</td>
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<td>0.543</td>
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<td>14</td>
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<td>9.000</td>
<td>10.000</td>
<td>9.000</td>
<td>0.077</td>
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<tr>
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<td>10.000</td>
<td>9.333</td>
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<td>8.000</td>
<td>8.667</td>
<td>0.064</td>
<td>0.556</td>
</tr>
<tr>
<td>18</td>
<td>C3.2.3</td>
<td>8.000</td>
<td>10.000</td>
<td>10.000</td>
<td>9.333</td>
<td>0.049</td>
<td>0.453</td>
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<tr>
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<td>9.000</td>
<td>0.010</td>
<td>0.086</td>
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<tr>
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<td>9.000</td>
<td>10.000</td>
<td>9.000</td>
<td>0.021</td>
<td>0.186</td>
</tr>
<tr>
<td>21</td>
<td>C3.3.3</td>
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<td>9.000</td>
<td>9.000</td>
<td>8.667</td>
<td>0.010</td>
<td>0.082</td>
</tr>
<tr>
<td>22</td>
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<td>8.667</td>
<td>0.008</td>
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<td>8.000</td>
<td>0.027</td>
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<tr>
<td>24</td>
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<td>9.000</td>
<td>0.027</td>
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<tr>
<td>25</td>
<td>C4.2.1</td>
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<td>9.000</td>
<td>9.000</td>
<td>8.667</td>
<td>0.015</td>
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</tr>
<tr>
<td>26</td>
<td>C4.2.2</td>
<td>8.000</td>
<td>10.000</td>
<td>9.000</td>
<td>9.000</td>
<td>0.015</td>
<td>0.137</td>
</tr>
</tbody>
</table>

\[ CS = \sum_{i=1}^{26} W_i \times ASC_i = 8.519, \text{ Class 3U, Serious structure deterioration. Urgent maintenance needed} \]

4. Conclusions

The method used for carrying out the expertise of this study is commonly known as the Fuzzy Hierarchical Analytical Process (FAHP). This approach allowed us to assess the current state of the masonry bridge (subject of this study). Conclusions from this work are presented as follows:

- It is difficult for experts to provide accurate numeric values from comparison reports. In addition, the decision-making process when appraising the health state of a structure such as masonry bridges is complex and uncertain. Among the factors contributing to this complexity is the ambiguity of information during a visual inspection. Adding to this, the large number of criteria affecting the priority of judgment. This certainly leads experts to make different judgments.

- The fuzzy hierarchical analytical process is needed as a reliable and efficient method to overcome the problem of uncertainty in expert judgments.

- Four-level hierarchical structure was sufficiently constructed, integrating 26 sub-criteria. Score results showed that the structural capacity gained the greatest impact on bridge deterioration (weight equal to 56.5%) followed by environmental conditions (weight equal to 22.5%). Factors of bridge history and professional involvement had the lowest impact (weight equal to 12.5%, and 8.5%).
The use of the IQOA grading system has been successfully combined. Indeed, using this classification system, it was possible to calculate a CS index which characterizes the degree of degradation of the structure. The obtained CS (8.519) located in the interval [8, 10] classifies the structure as highly degraded (Class 3U according to IQOA).

Bridge of masonry situated at Mohammadia was successfully expertized using FAHP approach, as has been proposed in this article. The bridge is classified in the category U3. Consequently, the structure presents a high structural risk and requires very urgent maintenance.

The FAHP method has demonstrated its effectiveness in eliminating the uncertainties and ambiguities present during an appraisal of masonry bridges. For this reason, this methodology is recommended for use in future survey applications to assess the condition of existing masonry structures in general.

5. Declarations

5.1. Author Contributions

Conceptualization, M.L. and A.M.; methodology, M.L., AM. and A.D.; software, M.L. and A.D. validation, M.L. and A.D.; formal analysis, M.L. and A.D.; investigation, M.L.; resources, M.L.; data curation, M.L.; writing—original draft preparation, M.L., AM. and A.D.; writing—review and editing, M.L. and A.D.; visualization, M.L. and A.D.; supervision, M.L. and A.D. 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


A Comparative Study on Soil Stabilization Relevant to Transport Infrastructure using Bagasse Ash and Stone Dust and Cost Effectiveness

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Abstract

Soft ground improvement to provide stable foundations for infrastructure is national priority for most countries. Weak soil may initiate instability to foundations reducing their lifespan, which necessitates the adoption of a suitable soil stabilization method. Amongst various soil stabilization techniques, using appropriate admixtures is quite popular. The present study aims to investigate the suitability of bagasse ash and stone dust as the admixtures for stabilizing soft clay, in terms of compaction and penetration characteristics. The studies were conducted by means of a series of laboratory experimentations with standard Proctor compaction and CBR tests. From the test results it was observed that adding bagasse ash and stone dust significantly upgraded the compaction and penetration properties, specifically the values of optimum moisture content, maximum dry density and CBR. Comparison of test results with available data on similar experiments conducted by other researchers were also performed. Lastly, a study on the cost effectiveness for transport embankment construction with the treated soils, based on local site conditions in the study area of Assam, India, was carried out. The results are analyzed and interpreted, and the relevant conclusions are drawn therefrom. The limitations and recommendations for future research are also included.

Keywords: Soil Stabilization; Admixtures; Stone Dust; Bagasse; California Bearing Ratio; Compaction.

1. Introduction

Reducing long-term settlement of infrastructure and providing cost-effective foundations with sufficient load-bearing capacities are national priorities for infrastructure development in most countries. In particular, transport infrastructure build on soft soil can cause excessive settlement, initiating the undrained failure of a super-structure if proper ground...
improvement is not carried out [1]. Adequate ground improvement techniques can be adopted to prevent unacceptable excessive and differential settlement and increase the bearing capacity of the foundations at low cost [2, 3]. Over several decades, various ground improvement techniques have been undertaken around the world, which include the use of admixtures, chemical stabilization, and dynamic compaction, as well as preloading with vertical drains and stone columns, among others [4, 5].

The most common and easily implementable soil improvement techniques for transport infrastructure are carried out either through mechanical stabilization or by using cementitious or non-cementitious additives, known as admixtures, in general, with the objectives of reducing compressibility, while enhancing the capacity and serviceability of the soil sub grade supporting the transport corridors.

The existing literature indicates that there are several methods of soft ground improvement using admixtures, including natural pozzolana, soil-quarry dust mixture, bagasse ash, granite dust, and lime, along with the use of marble dust, stone dust, and fly ash, with blends of wheat husk ash and sugarcane straw ash [6-8]. The suitability of various admixtures for ground improvement have been studied previously, including the use of bagasse ash with lime, cement stabilized soil with stone dust, and rice husk ash with polypropylene fiber [9-11]. However, the sole assessment of these additives to fulfill the strength and serviceability criteria of the treated soil implies that such admixtures are unsuitable for pavement subgrades [12-15]. Furthermore, when rice husk ash was used alone, an increase in vertical displacement took place [16]. Several researchers investigated the use of various other materials with bagasse ash, namely hydrated lime, amalgamated quarry fines, cement kiln dust, dolomite powder, and ordinary Portland cement, and found that such additions initiated curtailting of the soils’ plasticity index, although increases in soil bearing capacity were insignificant [17, 18].

In some cases, particle segregation and decreased soil strength were reported, especially when the addition of the stone dust as admixture was beyond 20-30% of the virgin soil’s dry weight, although its influence on the CBR values were not studied [19-21]. Zaika and Soeharjono [22] found that reduction in the value of soaked CBR took place while using bagasse ash alone as an admixture and suggested blending lime, Portland cement, and gypsum to enhance the CBR. Chen et al. [23] stated that the use of 2% lignosulphonate improved the shear strength of sandy silt and its ductile behavior. Blending the existing poor soil sub-grade with hydrated lime and bagasse ash managed environmental concerns through waste reduction [24]. Application of fly ash and quarry dust as admixtures exhibited significant decreases in void ratio, plasticity index, and swelling potential with shear strength increments of virgin soil [25].

The use of stone slurry containing lime as an admixture at a proportion of 4 to 5% enhanced the shear strength of virgin soil, although its influence on the soil’s bearing capacity was not studied [26, 27]. Ogila [28] investigated the decrease in swelling pressure and heave with the addition of ornamental limestone dust in samples of expansive soils, although alterations in the treated soil’s strength parameters were not observed. Hasan et al. [29] found that the presence of montmorillonite clay in soil treated with lime and bagasse ash was likely to initiate shrinkage cracks; in such cases, the use of geomembrane or an emulsified cushion was recommended.

1.1. Significance of the Research

For transport corridors, compaction and penetration characteristics are the two vital sub-grade soil properties to support transport infrastructure, such as the highways and railways. Although various admixtures to improve soft soil are available, as per the literature, the use of bagasse ash and stone dust have been quite effective, owing to local availability in large quantities, as well as low cost, besides satisfactory performance in soft soil stabilization. However, the available literature has yet to provide insight into a comparative investigation on the suitability of bagasse ash and stone dust as admixtures in terms of the compaction and penetration characteristics of treated soft soils, or a study on the cost effectiveness of such a soil stabilization technique, specifically for transport infrastructure. This present study aims to bridge this knowledge gap by conducting a comprehensive laboratory experimental program, followed by cost computations.

2. Experimentations

In this section, the materials used, their engineering properties, and the experimental approach and plan are described briefly in sequence.

2.1. Materials

The soft soil sample was stabilized by applying admixtures, i.e., bagasse ash and stone dust in target quantities. The material properties are described below.
2.1.1. Soft Soil

The soil sample used in this study for the laboratory tests was collected from Guwahati, Assam, India by means of an auger boring technique from a depth of about 1-2 m below the ground’s surface. The natural moisture content of the soil was measured about 31%. The sample was air-dried, and thereafter, used in the laboratory for investigation. The particle size distribution (PSD) performed by sieve analysis and hydrometer tests indicated that the soil could be classified as well-graded silty clay; the PSD curve is presented in Figure 1. The geotechnical properties are shown in Table 1. The soil may be classified as CL, after the unified soil classification system.

Limited research was carried out previously by other researchers with virgin soil at the study area around the Deepor Beel at Guwahati, Assam, India, including subsoil characterizations and groundwater quality assessment [30, 31], although any investigations on chemical stabilization with the virgin soil is yet to be conducted.

![Figure 1. Particle size distribution of untreated soft soil sample](image)

### Table 1. Geotechnical properties of untreated soil sample

<table>
<thead>
<tr>
<th>Geotechnical properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniformity coefficient ($C_u$); Coefficient of curvature ($C_c$)</td>
<td>27.5; 5.68</td>
</tr>
<tr>
<td>Atterberg limits *</td>
<td></td>
</tr>
<tr>
<td>Liquid limit</td>
<td>52%</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>19%</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>33%</td>
</tr>
<tr>
<td>Specific gravity of soil particles, $G$ **</td>
<td>2.64</td>
</tr>
<tr>
<td>Standard Proctor compaction test *</td>
<td></td>
</tr>
<tr>
<td>Optimum moisture content</td>
<td>19.34%</td>
</tr>
<tr>
<td>Maximum dry density</td>
<td>15.95 kN/m$^3$</td>
</tr>
<tr>
<td>California bearing ratio (CBR) **</td>
<td></td>
</tr>
<tr>
<td>Un-soaked sample</td>
<td>4.92%</td>
</tr>
<tr>
<td>Soaked sample</td>
<td>2.66%</td>
</tr>
</tbody>
</table>

* As per ASTM D4318 [53]; ** As per ASTM D5550 [54];
# As per ASTM D698 [35]; # # As per ASTM D1883 [36]

2.1.2. Bagasse Ash

The dry pulpy fibrous residue of sugarcane after juice extraction is termed as bagasse. It is extensively used as a building material, as well as for manufacturing biofuel [32]. The raw bagasse collected from sugar mills is oven dried, and thereafter burnt to ashes, which are used as an admixture for soil stabilization. The bagasse ash is dark black in wet conditions, and gray in dry conditions, consisting of Silica, as well as oxides of Magnesium, Calcium, Iron, Sodium, Potassium and Aluminium [22]. The bagasse is locally available in bulk quantities for utilization in ground improvement for transport infrastructure. The physical and chemical properties of the bagasse ash used for experimentation were obtained from the laboratories, and presented in Table 2. A representative photographic view of the bagasse ash is shown in Figure 2(a).

![Figure 2. Bagasse Ash](image)
2.1.3. Stone Dust

Stone dust, which is used as a civil construction material, is a waste material generated while crushing stones in a stone crusher that produces angular aggregates in different sizes. Stone dust is mostly reduced into powdered form after the breakdown of boulders and rocks and appears grayish in color. It is largely available in Guwahati and in other parts of Assam, India. A representative photographic view of the stone dust used in the experiments is shown in Figure 2(b). The particle size distribution curve (see Figure 3), obtained from sieve analysis data, indicated sand and gravel contents of 90 and 10% respectively. The geotechnical properties of the stone dust obtained from laboratory tests are given in Table 3.

Table 2. Properties of bagasse ash

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
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<tr>
<td>Specific Gravity</td>
<td>2.51</td>
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<tr>
<td>Blaine surface area</td>
<td>512 m²/kg</td>
</tr>
<tr>
<td>Particle size (D₅₀)</td>
<td>27.3 μm</td>
</tr>
<tr>
<td>Colour</td>
<td>Reddish Grey</td>
</tr>
<tr>
<td>SiO₂</td>
<td>64.73</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.96</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>7.56</td>
</tr>
<tr>
<td>CaO</td>
<td>12.67</td>
</tr>
<tr>
<td>MgO</td>
<td>3.23</td>
</tr>
<tr>
<td>SO₃</td>
<td>1.89</td>
</tr>
<tr>
<td>K₂O</td>
<td>3.96</td>
</tr>
</tbody>
</table>

Figure 2. Photographic views of: (a) bagasse ash, and (b) stone dust

Figure 3. Particle size distribution of stone dust
**2.2. Test Approach and Plan**

The virgin soil collected was first oven dried for 48 hours, and thereafter, manually ground and uniformly mixed with the above-mentioned admixtures at selected proportions by weight. Two different categories of stabilized soil samples (remolded), one with bagasse ash and the other with stone dust, were separately tested and comparative studies were carried out. While a standard Proctor test is suitable for ordinary transport infrastructure, pavements for heavier traffic loading, especially aircraft and frequent truck traffic, demand a modified Proctor test, following the procedure included in ASTM D1557 [33]. In the study area in Assam, India, the measured traffic loading is much lighter [34]. Hence, the standard Proctor test was followed.

The compaction characteristics of the stabilized soil samples were determined by a standard Proctor test following the procedure described in ASTM D698 [35]. On the other hand, the penetration characteristics of treated soil samples were determined by CBR tests for un-soaked and soaked samples, as per recommendation of ASTM D1883 [36]. To carry out the tests, the proportion of admixtures was varied between 2-10% by weight of the dry virgin soil sample and mixed separately with the soil. A total of 33 sets of tests were performed, including the untreated soil, as detailed in Table 4. To minimize the experimental error, three separate experiments were conducted for each set of tests, and the average values of the results were taken for analysis and interpretation. In the laboratory, the CBR specimens were prepared at the optimum moisture content for each test category. It is acknowledged the field CBR values may differ if the field moisture content is different from the optimum moisture content.

### Table 4. Test program

<table>
<thead>
<tr>
<th>Soil samples</th>
<th>Test category</th>
<th>Set of tests *</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated soil sample (i.e., no admixtures)</td>
<td>Standard Proctor compaction</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Un-soaked</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Soaked</td>
<td>1</td>
</tr>
<tr>
<td>Bagasse ash proportion by weight of untreated soil: 2, 4, 6, 8 and 10%</td>
<td>Standard Proctor compaction</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Un-soaked</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Soaked</td>
<td>5</td>
</tr>
<tr>
<td>Stone dust proportion by weight of untreated soil: 2, 4, 6, 8 and 10%</td>
<td>Standard Proctor compaction</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Un-soaked</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Soaked</td>
<td>5</td>
</tr>
<tr>
<td>Total set of tests =</td>
<td></td>
<td>33</td>
</tr>
</tbody>
</table>

* For each set, 3 separate tests were conducted to minimize the error, if any (variation in results in all test sets were observed to be less than 5%)

Both the bagasse ash and stone dust are largely available in bulk quantities around the entire study area in Assam, India at cheap rates because of the large number of sugar mills and rock quarries existing in the region [37, 38]. Previous studies revealed that the optimization of compaction and penetration characteristics was achieved when their quantities vary between 4-12% of the dry soil’s weight [10, 21, 29]. In order to ensure adequate enhancement of the treated soil’s compaction and penetration characteristics, while limiting the transport cost to ensure cost-effectiveness, the maximum quantity of admixtures was restricted to 10% in this paper.

It is true that based-on previous studies the bagasse ash needs to be treated with lime or cement as an activator, especially when the soil is expansive or very soft compressible clay [39]. In the present study, the virgin soil is silty clay. Thus, additional activators might enhance the cost significantly, compared to the relative benefits in the enhancement in the compaction and penetration characteristics. Hence, considering the local soil’s characteristics, an activator was not used for soil stabilization.
3. Results and Discussions

The plot of maximum dry density versus moisture content for untreated soil (i.e., without any admixture), obtained from the standard Proctor test data, is shown in Figure 4, from which the optimum moisture content and dry density were evaluated as 19.2% and 15.95 kN/m$^3$, respectively. For the CBR test for un-soaked and soaked untreated soil, the plot of applied plunger load versus the penetration is shown in Figure 5. After incorporating correction in the load-axis, the CBR values for un-soaked and soaked specimens were evaluated as 4.92 and 2.66%, respectively.

![Figure 4. Compaction curve for untreated soil](image)

![Figure 5. Load-penetration data for CBR tests of untreated soil](image)

The Proctor test results for the treated soil are presented in Figure 6. The CBR test results for un-soaked and soaked treated soil are shown in Figure 7 (with corrected load axis). The optimum moisture content, maximum dry density, and CBR values for the untreated and treated soils are included in Table 5.
3.1. Main Findings: Analyses and Interpretations

To study the influence of admixtures on the compaction and penetration characteristics of the virgin soil, the optimum moisture content, maximum dry density, and CBR value were normalized as follows (Equations 1 to 3):

\[
\alpha_o = \frac{\text{Optimum moisture content of stabilized soil}}{\text{Optimum moisture content of untreated soil}}
\]

\[
\alpha_m = \frac{\text{Maximum dry density of stabilized soil}}{\text{Maximum dry density of untreated soil}}
\]

\[
\alpha_c = \frac{\text{CBR of stabilized soil}}{\text{CBR of untreated soil}}
\]

where, \(\alpha_o\), \(\alpha_m\), and \(\alpha_c\) are the normalized values of optimum moisture content, maximum density, and CBR, respectively.

![Figure 6. Compaction curves for treated soil: (a) bagasse ash, and (b) stone dust](image)

![Figure 7. Load-penetration curves for CBR tests for treated soil: (a) bagasse ash, and (b) stone dust](image)

3.1.1. Optimum Moisture Content

The variation of normalized optimum moisture content (\(\alpha_o\)) with bagasse ash content is portrayed in Figure 8(a). As the bagasse ash content increased from 2 to 10%, the parameter \(\alpha_o\) was observed to increase from 1.12 to 1.47. The pattern of variation was found to be curvilinear with a descending slope. Figure 8(b) depicts the variation of \(\alpha_o\) with increasing the content of stone dust. The normalized optimum moisture content was observed to increase in the range of 1.09 < \(\alpha_o\) < 1.61 as the content of stone dust increased from 2 to 10%, the pattern of variation being relatively linear.
The value of $\alpha_o$ greater than unity indicated increases in the optimum moisture content due to the addition of admixtures, the value being slightly higher in the case of stone dust. The above observations may be justified by the possible occurrence of ion exchange between the admixtures and soil particles [40]. In addition, the admixture particles probably reduced the free silt and clay fractions in the soil, thereby occupying larger void spaces for water retention.

### Table 5. Test results

<table>
<thead>
<tr>
<th>Admixture type</th>
<th>Admixture content (%)</th>
<th>Standard Proctor compaction test</th>
<th>CBR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Optimum moisture content (%)</td>
<td>Maximum dry density (kN/m$^3$)</td>
</tr>
<tr>
<td>None (i.e., untreated soil)</td>
<td>0</td>
<td>19.2</td>
<td>15.95</td>
</tr>
<tr>
<td>Bagasse ash</td>
<td>2</td>
<td>19.41</td>
<td>15.82</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>23.98</td>
<td>15.64</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>25.22</td>
<td>15.53</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>26.14</td>
<td>15.41</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>27.45</td>
<td>15.23</td>
</tr>
<tr>
<td>Stone dust</td>
<td>2</td>
<td>20.21</td>
<td>15.95</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>22.67</td>
<td>15.90</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>25.19</td>
<td>15.86</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>27.32</td>
<td>15.75</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>30.12</td>
<td>15.61</td>
</tr>
</tbody>
</table>

3.1.2. Maximum Dry Density

Figure 9(a) presents the variation of normalized maximum dry density with bagasse ash content. As observed, the parameter $\alpha_m$ decreased fairly linearly in the range of $0.95 < \alpha_m < 0.99$ as the bagasse ash content increased from 2 to 10%. In the case of stone dust, on the other hand, as shown in Figure 9(b), $\alpha_m$ decreased following a curvilinear pattern with a descending slope with increasing stone dust content; the range of variation being $0.97 < \alpha_m < 1.0$. The values of $\alpha_m$ less than unity indicated a reduction in the value of maximum dry density due to admixture addition. This is advantageous in terms of the decrease in the self-weight of the subgrade with stabilized soil at optimum moisture content. Considering the findings of Kaniraj and Havangi [41], the above observation may be justified with the possible accumulation and flocculation of virgin soil particles with ion exchange between the admixture molecules, which probably initiated the weight-volume ratio reduction.

3.1.3. California Bearing Ratio

The variation of the normalized CBR against increasing bagasse ash content is plotted in Figure 10(a). The increasing bagasse ash content from 2 to 10% initiated the parameter $\alpha_c$ to increase in the ranges of $1.0 < \alpha_c < 2.2$ and $1.25 < \alpha_c < 3.25$ for the un-soaked and soaked tests, respectively. The patterns of variation in both the cases were observed to be curvilinear with ascending slopes; for the un-soaked tests, a reverse curvature was noted with a point of inflection at a bagasse content of about 5%.

Figure 10 (b) depicts the variation of the parameter $\alpha_m$ with the stone dust content. The range of variation of $\alpha_m$ was found to be $1.0 < \alpha_m < 2.7$ and $1.25 < \alpha_m < 2.3$ for the un-soaked and soaked tests, respectively. The pattern of variation was observed to be curvilinear with ascending slopes.
For both of the above cases, the value of the parameter $\alpha_c$ was found to be greater than unity, indicating enhancement in the CBR with respect to the untreated soil, due to addition of admixtures. Furthermore, the values corresponding to those with the stone dust were higher compared to those with the bagasse ash, which implies that the soil stabilized with stone dust produced lower penetration-susceptibility.

Figure 9. Variation of normalized maximum dry density, $\alpha_m$ with: (a) bagasse ash content, and (b) stone dust content

Figure 10. Variation of normalized CBR, $\alpha_c$ with: (a) bagasse ash content, and (b) stone dust content

Considering the findings of Mousavi and Karamvand [42], the above observation may be justified with the possible chemical reaction between the soil particles and admixtures. The admixtures probably attributed to cementing effects on the soil, increasing their penetration resistance, thereby increasing the CBR; such cementing efficiency appeared to be more in the case of stone dust. The untreated virgin soil had a specific gravity of 2.64, whereas the bagasse ash and stone dusts had specific gravities of 2.51 and 2.71, respectively. Therefore, mixing the virgin soil with the admixtures at various proportions altered the specific gravity of the treated soil. This factor attributed to the alteration in the treated soil’s CBR values [43, 44].

3.2. Comparison with Previous Studies

The test results obtained from the present study were compared with the previous experimental results of Sharma and Kaushik [10] and Zaika and Soeharjono [22] for bagasse ash test data, and Agarwal [20], Kumar and Bishnoi [26] and Venkateswarlu et al. [27] for stone dust test data, as shown in Figures 11 and 12.

3.3. Implications and Explanations

In the case of the standard Proctor compaction tests, the parameter $\alpha_o$ was observed to vary following a random pattern with increasing admixture content, in the case of previous test data, as opposed to a regularized manner for the current tests, as observed from Figure 11(a). The parameter $\alpha_m$, on the other hand (see Figure 11b), was observed to decrease with increasing admixture content in the case of previous test data; for bagasse ash, the pattern of variation was regular curvilinear, whereas for stone dust, it was random. In the case of previous test data relevant to the CBR tests, the parameter $\alpha_c$ was observed to vary in a random pattern (see Figure 12) against a regularized pattern for the current tests, as discussed above. The difference in magnitudes, as well as the pattern of variation for the parameters $\alpha_o$, $\alpha_m$, and $\alpha_c$ in the case of the previous test data compared to the current test results may be justified with the fact that the soil types were different, collected from various sites, in the cases of previous tests by other researchers. Moreover, the properties of the bagasse ash and stone dust used were also of different properties, compared to the present experiments.
Figure 11. Comparative studies with previous test data for: (a) $\alpha_o$, and (b) $\alpha_m$

Figure 12. Comparative studies with previous test data for $\alpha_c$
4. Cost Effectiveness

Highways are considered as nationally important and require periodical maintenance. In rural areas of Assam, India, constructing a pavement requires higher thicknesses of base course and sub-base course to provide adequate drainage facilities, which undoubtedly increases the construction cost. Basack et al. [45] investigated the cost effectiveness of fly ash in pavement construction, and it was observed that using fly ash reduced the cost significantly. The present study is an attempt to estimate the cost of pavement construction with and without additives. From the analysis, it is observed that using soil treated with bagasse ash in pavement construction is more economical in comparison to untreated soil and soil treated with stone dust (see Tables 6 and 7, as well as Figures 13 and 14). The analysis reveals that the cost, compared to the embankment constructed with untreated virgin soil, is reduced by 14.43 and 9.67 % in the cases of bagasse ash and stone dust at 10% proportions, respectively.

For designing the proposed pavement, the project requirements were established by analyzing the soil properties. The dimensions of the road and design parameters, such as design life and traffic estimations, were considered as per Indian standard technical specifications [46]. The embankment’s trial geometry was finalized following the recommended guidelines available [47]. This research is intended to stabilize soil along the stretches of Deepor Beel, a freshwater lake that forms a channel to the Brahmaputra River in Guwahati, Assam, India. As per the sample analysis, as well as the transportation and storage facility available at site, an optimum of 10% bagasse ash and 10% stone dust can be used for securing the banks of Deepor Beel significantly to stabilize the soil. Local availability of both bagasse ash and stone dust in bulk quantity is a major advantage. In addition, the optimum percentage of additives conforms to economic stabilization of pavement sub-grades, thereby curtailing the construction cost significantly.

The components of costs related to bringing additives to the site, time of treatment, and mixing costs are added in the analysis. The optimum time and temperature of the calcinations process to produce bagasse ash with high pozzolanic activity is three hours and a 600°C at 10°C per minute heating rate, respectively [48]. While there are various procedures of mixing of materials at the site, the mix-in-place method, which should be equipped with rotavator and compacting with a smooth wheel roller to achieve the desired density, is recommended from the available information based on the local conditions [49-52].

Table 6. Design criteria of embankments using untreated and treated soils

<table>
<thead>
<tr>
<th>Embankment Design Components *</th>
<th>Untreated soil</th>
<th>Treated with bagasse ash</th>
<th>Treated with stone dust</th>
</tr>
</thead>
<tbody>
<tr>
<td>C.B.R. value of subgrade (Soaked) **</td>
<td>2.66 %</td>
<td>8.53 %</td>
<td>7.05%</td>
</tr>
<tr>
<td>Pavement thickness</td>
<td>Sub-base course</td>
<td>200 mm</td>
<td>70 mm</td>
</tr>
<tr>
<td></td>
<td>Base course</td>
<td>150 mm</td>
<td>150 mm</td>
</tr>
</tbody>
</table>

Initial traffic (CVPD) # | 44 |
Design life | 10 years |
Growth rate factor | 6% |
Projected traffic (CVPD) | \( A = P \ (1+r)^{n+x} = 28 \times (1+0.06)^{10+10} = 51 \) |
Top width of embankment | 7.5 m |
Carriage way | 3.5 m |
Height of embankment | 2.0 m |
Side slope | 2H : 1V |
Bottom width of embankment | 15.5 m |
Side slope earth over thickness | 1.0 m |
Length of embankment | 1000 m |

* Design criteria are as per MORTH [46];
# Average number of commercial vehicles per day (rural roads);
** The soil samples are to be mixed with additives uniformly in fully submerged condition for 4 days before CBR test.
Table 7. Estimation of cost of construction using untreated and treated soils

<table>
<thead>
<tr>
<th>Material</th>
<th>Pavement Components</th>
<th>Quantity</th>
<th>Rate * (Indian Rupees **)</th>
<th>Amount (Indian Rupees)</th>
<th>Total amount (Indian Rupees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated Soil</td>
<td>Base course</td>
<td>0.15 × 8.0 × 1000 = 1200 m³</td>
<td>1,608 per m³</td>
<td>19.3 M **</td>
<td>2.5 M</td>
</tr>
<tr>
<td></td>
<td>Sub-base course</td>
<td>0.10 × 3.75 × 1000 = 375 m³</td>
<td>1,544 per m³</td>
<td>0.58 M</td>
<td></td>
</tr>
<tr>
<td>Soil treated with bagasse ash</td>
<td>Base course</td>
<td>0.15 × 8.0 × 1000 = 1,200 m³</td>
<td>1,608 per m³</td>
<td>19.3 M</td>
<td>2.1 M</td>
</tr>
<tr>
<td></td>
<td>Sub-base course</td>
<td>0.035 × 3.75 × 1000 = 131.25 m³</td>
<td>1,544 per m³</td>
<td>0.2 M</td>
<td></td>
</tr>
<tr>
<td>Soil treated with stone dust</td>
<td>Base course</td>
<td>0.15 × 8.0 × 1000 = 1,200 m³</td>
<td>1,608 per m³</td>
<td>19.3 M</td>
<td>2.2 M</td>
</tr>
<tr>
<td></td>
<td>Sub-base course</td>
<td>0.055 × 3.75 × 1000 = 206.25 m³</td>
<td>1,544 per m³</td>
<td>0.31 M</td>
<td></td>
</tr>
<tr>
<td>Bagasse ash *</td>
<td>Requirement</td>
<td>0.1 × 131.25 × 15.23 = 199.89 kN</td>
<td>51 per kN</td>
<td>10,194</td>
<td>0.01 M</td>
</tr>
<tr>
<td></td>
<td>Transportation and</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mixing</td>
<td>200 kN</td>
<td>20 per kN</td>
<td>4,000</td>
<td></td>
</tr>
<tr>
<td>Stone dust *</td>
<td>Requirement</td>
<td>0.1 × 206.25 × 14.78 = 304.83 kN</td>
<td>112.96 per kN</td>
<td>10,194</td>
<td>0.01 M</td>
</tr>
<tr>
<td></td>
<td>Transportation and</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mixing</td>
<td>305 kN</td>
<td>25 per kN</td>
<td>7,625</td>
<td></td>
</tr>
</tbody>
</table>

Summarized total cost (Indian Rupees)

- Untreated virgin soil: 2.5 M
- Soil treated with bagasse ash: 21,32,250 + 14,194 = 21,46,444 M
- Soil treated with stone dust: 22,48,050 + 17,819 = 22,65,869 M

Saving of cost per 1000 m construction: 14.43% or 9.67%.

*As per APWRD [51]; ** M stands for million;
* As per local market rates; ** 1 Indian Rupee = 0.013 US Dollar

Figure 13. Site location of study area
5. **Summary and Conclusions**

A laboratory-based investigation was performed with the objective of stabilizing soft soil by the addition of admixtures, namely bagasse ash and stone dust; the admixture content was varied from 2-10%. The compaction and penetration characteristics of the treated soil were studied through a series of standard Proctor and CBR tests.

The paper presents a comprehensive study on the suitability of using bagasse ash and stone dust as admixtures for soft ground improvement, in terms of compaction and penetration characteristics. Through extensive laboratory experimentations, the variation of optimum moisture content, maximum dry density, and soaked and un-soaked CBR with admixture content were studied in detail. Through appropriate costing analysis, based on local conditions, the proposed soil stabilization technique was found to be quite cost effective in the case the soft ground supports for transport infrastructure. However, it is essential to conduct a generalized study on the improvement of soft soil in terms of strength, stiffness, and durability [57]. Moreover, the movement of vehicles via transport corridors initiates cyclic loading on the soil sub-grade, altering its strength and stiffness [58]. This study aspect was not covered in the paper.

The study revealed that the optimum moisture content of the stabilized soil increased, in comparison with that of the untreated soil; the increment is up to 47 and 61% in the cases of bagasse ash and stone dust, respectively. While the variation pattern of the optimum moisture content with the admixture content is curvilinear for bagasse ash, the same is observed to be linear in the case of stone dust.

The maximum dry density of the stabilized soil decreased, compared to that of the untreated virgin soil, up to about 5% for bagasse ash and 7% for stone dust. The pattern of variation is linear for bagasse ash and curvilinear in the case of stone dust. The addition of admixtures produced significant enhancement in the soil’s CBR; the increment being as high as 120 and 225% of that of the untreated virgin soil, for un-soaked and soaked samples, respectively, with bagasse ash used as the admixture. In case of stone dust, the increment was observed to be 170 and 120%, respectively. The pattern of variation was observed to be curvilinear.

A comparative study to justify the suitability of the two different admixtures implies that bagasse ash produced relatively less increment in the optimum moisture content, and almost an identical decrease in the maximum dry density, compared to stone dust. This indicates comparatively lower water requirements for initiating compaction in the case of bagasse ash. Additionally, the use of bagasse ash enhanced the soaked CBR significantly, thereby implying a higher penetration susceptibility, compared to the stone dust.
As far as cost effectiveness is concerned, the use of bagasse ash and stone dust can reduce costs significantly. Hence, the appropriate admixture choice would depend on several other factors, including local availability and site treatment procedure costs.

5.1. Recommendations for Future Research Directions

As discussed above briefly, future research should be directed towards a generalized study on the utility of bagasse ash and stone dust as admixtures for chemical soil stabilization. Research should also consider the short-term and long-term enhancement of strength, stiffness, and durability of the treated soil through in-situ and laboratory tests, including plate load tests, vane shear tests, unconfined compressive and tri-axial tests, consolidation tests, etc. In addition, shrinkage and swelling tests and ductility tests should also be conducted for expansive soils. To ascertain the cyclic characteristics of the treated soil in the case of transport infrastructure, cyclic direct shear or tri-axial tests should be conducted. Lastly, a more general cost analysis involving other types of structures may be conducted as well. Such generalized study is currently under progress by the authors, and interesting results are expected.

6. Nomenclature

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\alpha)</td>
<td>Normalized values of CBR</td>
</tr>
<tr>
<td>(\alpha_{on})</td>
<td>Normalized maximum dry density</td>
</tr>
<tr>
<td>(a_{opt})</td>
<td>Normalized optimum moisture content</td>
</tr>
<tr>
<td>(A)</td>
<td>Number of commercial vehicles per day for design</td>
</tr>
<tr>
<td>(C_u)</td>
<td>Uniformity coefficient</td>
</tr>
<tr>
<td>(G)</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>(n)</td>
<td>Number of years between the last count and the year of completion of construction</td>
</tr>
<tr>
<td>(P)</td>
<td>Number of commercial vehicles per day at last count</td>
</tr>
<tr>
<td>(r)</td>
<td>Annual growth rate of commercial traffic</td>
</tr>
<tr>
<td>(x)</td>
<td>Design life in years</td>
</tr>
</tbody>
</table>

7. Declarations

7.1. Author Contributions


7.2. Data Availability Statement

The data presented in this study are available in article.

7.3. Funding

Minor financial support to procure materials, as well as carry out the laboratory experiments, data collection, and infrastructure support was received from the Scholar’s Institute of Technology and Management, Elitte College of Engineering, and University of Nevada, Las Vegas. However, no formal funding sources were received by the authors to conduct the research; hence information such as, Funder, Award Number, and Grant Recipient are not available.

7.4. Acknowledgements

The authors thankfully acknowledge the assistance received from Mr. Dhruba Bhagaboti, who was in-charge of the laboratory in the Department of Civil Engineering at Assam Engineering Institute during the experiments.

7.5. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Mangroves As Coastal Bio-Shield: A Review of Mangroves Performance in Wave Attenuation

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Abstract

Mangroves have been recognized as soft structures that provide coastline protection. The capability of dampening waves helps minimize destruction from catastrophic events including erosive wave attacks, torrential storms, and tsunamis. Mangroves act as the first line of coastal defense in natural tragedies such as during the Super Typhoon Haiyan 2013 and Indian Ocean Tsunami 2004, whereby the leeward mangrove area encountered less damage than the unprotected area. This has further brought the attention of researchers to study the attenuation performance of these coastal vegetations. Based on an extensive literature review, this paper discusses the attenuation mechanism of mangroves, the factors influencing the dissipation performance, studies on mangrove dissipation via different approaches, the dissipation efficiency, mangrove conservation and rehabilitation efforts in Malaysia and implementation of mangrove as coastal bio-shield in other countries. The study highlights that mangrove parameters (such as species, width, density etc.) and wave parameters (such as wave period and incident wave height) are among the contributing factors in mangroves-induced wave attenuation, with different efficiency rates performed by different mangroves and waves parameters. Towards that end, several improvements are proposed for future research such as to incorporate all influencing dissipation factors with specific analysis for each species of mangroves, to perform validation on the studied mangroves attenuation capacity in different settings and circumstances, as well as to address the extent of protection by the rehabilitated mangroves. A systematic and effective management strategy incorporating ecological, forestry, and coastal engineering knowledge should be considered to ensure a sustainable mangroves ecosystem and promising coastline protection by mangroves.

Keywords: Mangroves Ecosystem; Wave Dissipation; Coastal Protection; Rehabilitation; Conservation.

1. Introduction

Mangroves are distinctive ecological ecosystems among those situated between the land and sea along tropical and subtropical coasts. The mangrove coastal vegetations exhibit life-history adaptations to the challenges of both difficult establishment in mobile due to current dynamics and ocean wave influence with high salinity (0-90 degrees/thousand) in aqueous, anoxic sediments [1]. Nonetheless, mangroves provide coastal protection by attenuating wave height and energy, acting as a natural barrier to incoming waves and, therefore, reducing erosion [2]. Globally, mangroves are distinguished into two regions, the West, and the East. The West includes the African coasts of Atlantic, North, and South America, and the East incorporate the African coast. Studies have shown that the East zone represents the higher species richness [3]. According to Spalding (1997), 18,100,000 ha of mangroves have been reported in 1997 [4], which

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The affected areas in India during the IOT 2004, Pichavaram and Muthupet revealed that dense mangroves protected the areas with fewer casualties and less property damage recorded [16]. In Sri Lankan village, the densely populated mangroves areas caused only two deaths, meanwhile 6,000 people were found dead in area with no mangrove’s protection [17]. Nevertheless, the protection function it provides is not only relevant for tsunami cases but also applicable for other natural calamities such as storm surges and cyclones [18, 19] where the areas with mangroves were less damaged compared to mangrove-free areas [20]. The super-cyclone that struck Orissa, India in 1999 left 7.5 million people homeless with approximately 10,000 death tolls, except those protected area behind the healthy mangroves that suffered less losses [7]. In Philippines, mangroves safeguarded from the great waves impact of Super Typhoon Haiyan in 2013 [21]. These events, apart from similarly evident the buffering capacity of mangroves, have also sent a vivid message that conservation and sustainable management of these coastal vegetations are important to guarantee secure barrier against the natural hazards.

The complex root system, canopy, trunk, and few other geometries attributed by mangroves play vital role in reducing the severe effect of incoming waves, winds, and storms. Although the protection provided seems obvious, but the mechanism and process involved might not be well described. The shielding performance might differ according to various influencing factors too. This paper hereby attempts to review and discuss on wave attenuation mechanism by mangroves, the influential dissipation factors, previous studies on mangrove-induced wave dissipation via field assessments, numerical modeling, and laboratory studies, and the efficiency of dissipation based on extensive literature review. Mangrove conservation and rehabilitation efforts in Malaysia and the mangroves bio-shield implementation in several countries are also highlighted in this paper.

2. Research Methodology

The methodology adopted in this paper is simplified in the flowchart as shown in Figure 1.

The literature search was performed in Research Gate and Science Direct databases to retrieve available related papers. Several keywords were applied as search criteria, such as mangroves, mangroves ecosystem, coastal ecosystem, coastal protection, wave dissipation, wave attenuation, mangroves rehabilitation, mangroves conservation, mangroves degradation, Indian Ocean Tsunami, cyclone, typhoon, mangroves Malaysia, and coastal bio-shield to capture papers published before August 2021. Papers obtained were then filtered to sort only the most relevant information and data concerning the topic discussed. Subsequently, papers were classified into few categories. An evaluation and analysis on the literature was critically conducted, followed by the process of synthesizing the critical analysis, organizing the reviewed papers in a summary table according to the specific topic and finally writing process.
3. Wave Dissipation by Mangroves

Waves are attenuated by disturbances unraveling at a reduced depth of water within mangrove forests and resistance by mangrove roots, stems, and tree canopies in adequately deep water. Wave energy dissipation caused by wave deformation is merely a function of wave parameters (height, length, and period) and depths, especially in intertidal zone morphology and can be estimated numerically if these parameters are known [1, 22]. According to Parvathy and Bhaskaran (2017), steep slopes’ shoal distances become short, while the reduction in the height and part of surface waves may be reflected from the steep bottom [23]. On the other hand, mild slopes have a longer wave traveling distance and the waves will decay on mild slopes via the mangroves.

Wave dissipation in the vicinity of mangroves occurs due to the drag force and bottom friction. Drag force is normally associated with the resistance imposed by mangrove structures such as trunks, roots, and canopies (in cases where mangroves are fully submerged). Bottom friction, on the other hand, is resulted from the bed roughness. Both force and friction act in the opposite direction from the incoming wave (refer to Figure 2).

![Figure 2. Wave attenuation by mangroves](image)

The drag friction slows down the wave motion’s propagation into the mangrove forests [24, 25, 26]. The waves subsequently lose part of their energy and, therefore, attenuating them. When waves travel through the forests, the incident wave height is reduced due to bottom friction and the drag force exerted. The bottom friction alone might be insufficient to attenuate the wave height [27]; hence, mangroves are needed to enhance the force for a net reduction.

Differences between the height of incident waves (H_i) and transmitted waves (H_t) over the distance travelled inside the mangroves is defined as the wave reduction rate (r). Hence, it is important to calculate the rate of reduction. Equation 1 was established by Mazda et al. (2006) for the reduction rate calculation [28]:

\[
r = \frac{-\Delta H}{H} \cdot \frac{1}{\Delta x}
\]

Where \( r \) is the wave height reduction rate per unit distance (m\(^{-1}\)), \( \Delta H \) is the reduction in incident wave height (m), \( H \) is the incident wave height (m), and \( \Delta x \) is the distance travelled over the mangroves (m). The reduction rate coefficient as developed by Rasmeebsamuang and Sasaki (2015) is expressed in Equation 2 to represent the dissipation rate of waves [29], as follows:

\[
R(\%) = \left( \frac{H_i - H_t}{H_i} \right) \times 100
\]

Where \( R \) is the coefficient of wave reduction (%), \( H_i \) is the incident wave height (m), and \( H_t \) is the transmitted wave height (m). Equation 1 differs from Equation 2 such that the distance or width of mangroves is introduced in Equation 1, whereas Equation 2 only addresses the reduction waves. Previous studies revealed that the rate of wave attenuation largely depends on the density of mangrove forests, especially the diameter of the mangrove roots and trunks, and on the spectral characteristics of the incident waves [30].

4. Factors Governing Mangroves Performance in Dissipating Wave

Wave reduction in mangroves occurs due to several factors that can be divided into three categories as follows: 1) mangrove characteristics such as the width, density, species, root diameter, age, and canopy, 2) wave parameters such as wave period and incident wave height, and 3) other external factors such as bathymetry and water depth. Thus, understanding the factors that lead to the wave reduction mechanism is crucial for the management of mangrove areas to protect the coastline. The findings explained that the attenuation performance rate is significantly depending on the
parameters of mangroves, waves, and other external factors with linear relationship between the parameters and attenuation are observed in width, structures, density, age, and bathymetry, while water depth is found negatively correlated with dissipation of waves. Relationship with species and incident wave height are dependable on their types of species and wave period.

4.1. Width

The sea wave height decays exponentially as the width of mangroves increases. Studies have found that wave amplitude has an inverse relationship with the width of mangrove forests. According to Lee et al. (2021), the percentage of wave height reduction by mangroves is more than 60% over a 500 m width [2]. On the coast of Vietnam, Bao (2011) confirmed the reduction in wave height over the increment of distance into mangroves [31]. As the waves travel further into the mangroves forest in an increasing distance, more obstruction and interaction occur between the incoming wave and the friction exerts by both mangroves drag coefficient and seabed roughness [32], thus resulting to more losses of wave energy and height.

Considering that mangrove width has a significant role in the reduction of wave intensity, Shahruzzaman (2018) suggested a replantation of minimum width required at their study area so that mangroves can provide adequate coastline protection [33]. The study also highlighted that without sufficient width, an optimum buffering capacity would not be guaranteed even with high vegetation index (high mangroves density, matured mangroves, etc.). Besides, Adytia and Husrin (2019) reported that the mangrove width with four times the wavelength of incident waves is required to fully attenuate the incoming wave height [34].

4.2. Species

Mangroves of different species (such as Avicennia, Sonneratia, Rhizophora, and Bruguiera) perform differently in attenuating waves according to the characteristics of each species. It is well-known that the Rhizophora species are most effective than other mangrove types. Their attenuating proficiency is due to the complex aerial root structures with greater friction to incoming waves that leads to a higher drag coefficient. The attenuation performance of Rhizophora was 57.73% on porosity of 0.9828 [35]. Hashim and Catherine (2013) also reported that 80% of waves can be attenuated by an 80 m wide Rhizophora [36]. Meanwhile, in a 100 m wide Sonneratia located in northern Vietnam, Mazda et al. (2006) found that up to 50% of wave energy can be reduced [28].

According to Muliddin et al. (2014), a minimum of 79% of waves were attenuated over 1 tree/m² Sonneratia in their numerical modeling [37]. Additionally, a study by Herison et al. (2017) showed that Avicennia can produce an attenuation rate of 0.24 m/km in mangrove forests ranging from 0.5 m to 3 m in height [38]. Besides, Horstman et al. (2014) tested wave dampening in Sonneratia, Avicennia and Rhizophora dominated areas, which demonstrated an attenuation rate of 0.002 m⁻¹ in Sonneratia and Avicennia forests with low density and 0.012 m⁻¹ in denser Rhizophora [39]. In this case, although the attenuation rate was marked higher in Rhizophora forest, but the density acts as another manipulated variable which made them incomparable in terms of dissipation performance due to species.

4.3. Structure

The amount of energy dissipated on the mangrove structures is influenced by factors such as the arrangement of stems, roots, and branches as well as submerged parts of the vegetation. In addition, stem stiffness can also contribute to wave dissipation rates [36]. Wave height along the propagation direction decreases non-linearly with the growth in the wave travel distance due to relation with higher height and stem density, and larger diameter plants [40]. Rasmeemasmuang and Sasaki (2015) has shown a relationship between hydrodynamic factors and botanic factors in wave reduction towards Rhizophora systems [29]. Wave energy transmission reduces when the number of trees increases.

In Vietnam, Tusinski and Verhagen (2014) claimed that mangroves’ emerging canopy has the highest efficacy in the decaying process of waves compared to the roots, trunks, and submerged canopy [27]. Mazda et al. (2006) also found that thick mangrove leaves have an influence on attenuation rates with a condition that the water depth was high enough to enable the submergence of the leaves [28]. However, Lee et al. (2021) reported different results in Singapore, whereby wave reduction by mangrove roots was 85%-100%, and trunks resulted in 94% of vegetation drag force [2]. Mangrove roots are also the main contributor to the vegetation drag force that increases up to 0-35% under storm conditions. Roots are the most efficient dissipation of wave energy when it comes to shallow water [41].

Bare land, as opposed to mangrove-covered areas, was observed to be less impactful in attenuating the wave height [36]. Teh et al. (2009) found that a 500 m mangrove width in Penang, Malaysia in tsunami wave conditions imposed a reduction ratio of 0.50 compared to 0.55 in the mangrove-free area [14]. This is parallel with the findings reported by Quartel et al. (2007) where the unvegetated mudflat relies only on the bottom friction for wave dampening [25]. This
condition demonstrates lower wave reduction due to the absence of additional friction and drag force exerted by the mangrove structures that impeding the flow.

4.4. Density

Previous studies discovered that high mangrove density will impose higher wave dissipation [19, 25, 31, 42]. In a denser mangrove forest, the gaps between the roots and trunks are minimized. Therefore, wave and root-trunk interaction are dominant and increases the tendency for the reduction of wave height [36]. Wolanski (2006) similarly claimed that the densely populated mangroves form the dense interlocking arial roots, thus reducing the porosity and increasing obstruction to the incoming wave [43]. Furthermore, the drag force from these vegetations helps dissipate the wave magnitude. Findings from Limura and Tanaka (2012) that examined a vegetation model with different densities and its effect in mitigating tsunami wave impact [44] also supported the hypothesis.

The dissipation capability of disturbed mangroves was carried out in the coastline of Singapore by Lee et al. (2021), where they found that the reduction of wave height was intense with the increasing density [2], while Lou et al. (2018) observed a lower transmission coefficient of waves in a denser mangroves forest; however, this was rather significant for deep water than in intermediate wave conditions [45]. All above-mentioned studies concluded the same findings that mangroves density and wave attenuation have a positive correlation in various state of wave conditions. In less dense mangrove forests, the effect of wave breaking plays an important role in wave attenuation [46].

4.5. Age

The age of mangroves refers to the trunk diameter, size of the tree, stem density, and root diameter [36, 47], while according to Latief and Hadi (2006), mangrove age is associated with the size of this vegetation [48]. Alongi (2008) and Danielsen et al. (2005) claimed that, as the age of a tree increases, the higher its resistance to wave destruction [42, 49]. This is due to the high diameter of trunks and firm roots of the matured tree. Meanwhile, younger mangroves can be easily uprooted by erosive waves. Younger mangroves were found to unsuitably withstand extreme and higher waves, resulting in washing away because they are easily uprooted [27]. Hence, due to their weak characteristics, younger mangroves require support such as geo-bags for wave-breaker and fibre-rolls for stabilizing during re plantation [50].

4.6. Bathymetry

Higher wave energy attenuated in coastal regions changes directly towards the bathymetric profile [23]. The bathymetric condition also influences the size of waves with an increase in depth along the distance from the coastal regions. Besides, a steeper slope promotes better wave height reduction [27]. This is due to the wave shoaling effect in the steep slope whereby less or no such effect would result in a mild and gentle slope. Current available studies on bathymetry influence are very limited in mangrove-induced wave dissipation scope, hence further research on this is suggested to get a clearer and robust conclusion on the relationship between both parameters.

4.7. Water Depth

The effect of water level on wave attenuation was examined in storm conditions. The wave height reduction rate with elevated water levels during high tides (0.001-0.005m-1) was smaller compared to the elevated water level only (0.002-0.035 m-1). Lee et al. (2021) stated that lower water level creates more turbulence in the bottom layer as proportion to the shallow water depth and influences more water motion [2]. But, in a deeper water depth, the water particles will be less affected by the obstacles, hence causing to less attenuation [51]. Mazda (2006) however hypothesized that higher wave height reduction occurs when the resistance coefficient and water depth are increasing at the same time due to the larger submergence of mangrove branches and leaves [28].

Findings from field experiment conducted by Quartel et al. (2007) was very similar [25]. They reported that the resistance due to the unvegetated sandy bed with increasing water depth resulted in a lower wave height reduction. Meanwhile, in the presence of mangroves forests, the resistance coefficient increased with the increasing water depth, resulting in a higher wave height reduction due to the larger submerged part of mangrove branches and leaves that clogs the water flow. According to Parvathy and Bhaskaran (2017), if the water depth is higher in the steep slope region, the waves attenuate the fastest as they travel through the roots, resulting in more drag resistance [23].

4.8. Incident Wave Height

In hydrodynamic conditions, incident wave heights are decreased due to wave breakage [52]. High wave heights cause wave breakage and produce drag force through the mangroves. In addition, the percentage of drag force due to breakage increases by less than 1%. Previous study has shown that mangroves are more effective in attenuating short period waves, compared to the longer ones (e.g., swell waves, tsunami waves, storm surges) [53]. Short period waves
dissipate better even in a narrow strip of mangroves, in which such condition might not be enough for protection from long period wave [26]. In contrary, Brinkman (1997) justified that wave attenuation is independent on the incident wave height. Either short or long period waves, they dissipate at a similar rate [54].

5. Mangroves Dissipation Performance

Previous studies on wave dissipation by mangroves can be classified into several approaches, including laboratory experiments, field assessments, and integration of laboratory and field works with numerical studies. Some numerical studies incorporated simulation and modeling with validation from laboratory and field assessment, while others only adopted numerical assessment such as regression analysis. This proves the vegetation-induced wave dissipation ability of mangroves with a variety of affecting factors being tested over different wave conditions and mangrove species.

5.1. Laboratory Experiments

A laboratory experiment by Hashim and Catherine (2013) explained the effect of different mangrove densities and tree arrangement in dampening wave height [36]. Denser mangrove forests resulted in higher wave reduction, while staggered and tandem arrangements demonstrated only a slight difference in wave attenuation. Besides, mangrove areas showed twice wave reduction compared to non-vegetated areas. The presence of mangroves also exerted greater drag force, contributing to greater energy loss.

Similar laboratory studies focusing on density were also carried out by Pasha and Tanaka (2016, 2017), which highlighted that greater density of emergent vegetation attenuated wave energy more effectively [55, 56]. The effect of the opposing and following current on the dissipation of waves influenced by emergent rigid vegetation was also investigated [57]. The study examined the current with manipulated velocity, water depth, and vegetation density on wave dissipation by coastal vegetation with a larger velocity ratio range. Numerical modeling was implemented to ensure feasibility due to the limitations of maximum generated stable current velocity in experimental works.

Additionally, Kristiyanto and Armono (2013) carried out a study to analyze the relation of wave steepness in the wave dissipation process via both laboratory works and field assessment [35]. Wave steepness was described in terms of wave height and wave period. Data were analyzed using regression analysis in deriving the wave attenuation formula. The ability to dampen waves, known as transmission coefficient (Kt), was determined through the difference in height between the incident wave and transmitted wave. Compared to the above-mentioned studies which experimented the normal wave and current, Strusinska-Correa (2013) tested the attenuation performance over different widths in different tsunami wave conditions [58]. The results showed that reduction was higher in a wider mangroves forest due to the longer distance travelled.

In laboratory studies, the mangrove model representation may result to different values of data depending on the physical characteristics of the mangrove model. For instance, some experiments duplicate the mangrove solely in the form of cylinder, which disregard their important structures like branches, roots, and canopies. Although the findings or theories are still valid and proven, but it might slightly affect the accuracy in the observed values of wave attenuation. Thus, the model with actual resemblance of mangrove should be rather considered. Other than that, the bed friction in the wave flume is usually ignored. Bed properties should be ensured to have almost similar coefficient as the muddy area in the vicinity of mangrove too.

5.2. Numerical Modeling

Numerical studies on wave attenuation are extensive. The vegetation model is usually described as coastal vegetation in general, yet the model is still applicable to mangrove cases. For instance, Limura and Tanaka (2012) performed modeling to elucidate the effects of varying coastal vegetation density on tsunami energy reduction [44]. Boussinesq-type equations were used by including porosity and resistance terms to resemble the drag force by mangroves.

An experiment was conducted to validate the simulation with a 10% error whereby future improvement of numerical model was required on the back row of vegetation and the boundary between different densities. Water level and velocity reduction were greater as the density increased in both uniform and combined arrangement of vegetation models. The mitigation effects of mangroves on tsunami wave energy, height, and velocities were also analyzed by Teh et al. (2009) [14]. Morison equation was incorporated in the modeling to represent friction provided by the mangroves. This study inputted the mangrove geometries data in Penang, Malaysia, into the run-up model TUNA-RP. The reduction ratios for given velocities and wave heights were found to vary significantly depending on the wave and mangrove parameters.

A similar model as Limura and Tanaka (2012) [44] was optimized by Adyitia and Husrin (2019) in describing the non-linear transformation of tsunami wave attenuation by mangroves [34]. They included an additional term of dissipation due to bottom roughness in the momentum equation. The relation between the required mangrove width
over the magnitude of wavelength to produce the respective dissipation rate was simulated. In contrast to long period wave, Van Rooijen et al. (2015) clarified the effect of vegetation in reducing short waves, infragravity waves, and mean flow using XBeach model [59], which was extended with formulations by Mendez and Losada (2004) to account for the wave attenuation by coastal vegetation [60].

Hu et al. (2014) later carried out modeling that enables the quantification of vegetation drag coefficient in current—wave conditions [61]. They tested the attenuation performance in a tidal current, which is often neglected in most studies. However, they estimated that steady current may lead to higher or lower wave attenuation, depending on the velocity ratio. In high and low tides events, mangroves shown higher attenuation ranging from 96% to 97% in high tides, and only 85% to 90% was observed during low tides [62]. The mangrove canopy and root system play a prominent role in reducing the wave height during high and low tides, respectively. Dalrymple empirical model was used with the integration of the forward differencing method which simulated mangroves as non-homogenous forest characteristics that most likely resemble the real mangrove forest.

Abdullah et al. (2019) modeled the effect of wave and mangrove parameters by adopting the Mansard-Funke method and spectral analysis [63]. Wave amplitude, wave period, and mangrove density were studied in terms of their sensitivity in wave energy dissipation. Dissipated wave energy was higher in a smaller wave period with more dissipation over denser mangroves in submerged conditions. The differences between wave heights reduction in different salinity zones, with and without vegetation and mud inputs were observed in Indian Sundarbans (IS) [64]. The study solved other literature gaps which usually assessed mangroves as one general species, whereby in this study all four different dominant mangroves species were encompassed. The output suggested that higher wave attenuation was observed in the hyposaline stations of western IS than to the hypersaline central sector.

Rigid vegetation represented by three types of vegetation models was tested in terms of their wave attenuation [65]. Genetic Programming (GP), Artificial Neural Networks (ANNs), and a laboratory experiment were adopted. More recent studies have also assessed mangrove-induced wave attenuation by treating mangroves as flexible vegetation [66]. The XBeach model, which was commonly associated with wave attenuation by rigid vegetation modeling, was simulated with the flexible vegetation dynamic model. It was proven that modeling is reliable in predicting wave dissipation by flexible vegetation.

Recognizing the advancement in numerical model, assessment of flexible vegetation by waves should gain considerable attention of researchers. It defeats the gap in rigid vegetation assessment that may not address the motions and forces of vegetation. Therefore, more comprehensive result can be achieved in understanding the dynamics driven by mangrove while assessing for their attenuation performance.

Additionally, drag coefficient or Reynolds number and Manning’s roughness coefficient are crucial elements in numerical modeling. They represent the frictions from the mangrove structures and seabed which mainly influence the dissipation of waves. Some numerical modeling made only assumptions on the coefficient value or taking the most relevant value from existing coefficients, whereby in reality the coefficients obviously vary according to several factors such as density and species. Future research on the accurate estimation of drag coefficient and Manning’s roughness need to be studied in order to reduce this uncertainty.

Another important improvement for numerical modeling is to get more scenarios and conditions to be validated using the model. This is because their study may conclude a strong finding for the specific coastal conditions, topographies, and wave conditions that they simulated only. For more holistic and relevant conclusions on the mangrove protection performance, it is then suggested to evaluate the numerical model which is to be ran and assessed across various conditions and scenarios.

5.3. Field Studies

Almost all waves studied via field approaches are wind-driven waves because of the difficulty in assessing and measuring storm surges or tsunamis conditions. Field studies on the role of mangroves in combating wave energy are numerous with various affecting variables. For instance, Quartel et al. (2007) conducted an assessment in the Red River Delta, Vietnam, to compare the attenuation in the presence and absence of mangroves [25]. The Kandelia candel structures such as trunks and roots were emphasized as an additional factor that gave extra drag force compared to bare land. Other than that, Mazda et al. (2006) also discussed the difference of wave attenuation with and without mangroves [28]. However, this study is only limited to the Sonneratia species, which possess different types of roots compared to other species, therefore resulting in different attenuation rates.

A study conducted by Bao (2011) on wave attenuation has been widely used in research related to the adequacy of mangroves in dissipating waves [31]. Field data collected in coastal Vietnam were post-processed and developed into an exponential term incorporating almost all affecting factors including wave parameters and mangrove characteristics. Apart from that, this study was not solely subjected to mono-species mangroves but was rather applicable to all four mangroves of dominant species.
In the coastal waters of Jakarta, Indonesia, the *Avicennia marina* species were evaluated by Herison et al. (2014) with the forest width taken as the manipulated variable [67]. They produced a formula describing the wave attenuation in terms of mangrove width and energy. In an extension of this study, Herison et al. (2017) conducted a similar study with a field data collection in East Lampung Regency, Indonesia [38] later in 2017. They examined another variation of mangrove width and exponential functions were developed on the relationship between mangrove width and wave attenuation.

However, the drawback in both studies lies in the limited scope of wave dissipation-governing factors assessed, in which only forest width was considered in the determination of attenuation performance. Other affecting factors were collected as mentioned in their method; however, these factors were not well-presented in the result and discussion sections as the focus was only on the width of mangroves.

Ismail et al. (2019) studied wave attenuation and mangrove density in terms of root density [68]. The *Rhizophora* species attributing to complex aerial root system was studied. Most studies commonly analyze the effect of root densities on a horizontal basis; however, this study also investigated the effect of vertical density. Horizontally, the root density over certain mangrove widths was determined. On the other hand, vertically, the root density was measured from the bottom towards the top vertical layer. Both density influences on wave attenuation were observed.

While the findings may provide coherent results that support the theories of wave dissipation by mangroves, the assessment should be validated in other different locations too, considering the different setting, hydrodynamics, and wave conditions at each location. It is recommended to ideally conduct the similar field studies to compare the dissipation ability of mangroves across other conditions so that their applicability in other scenarios and circumstances can be addressed for a robust conclusion.

One obvious gap from the previous studies can be seen in the limited scope of wave dissipation governing factors assessed, in which only forest width and density were mostly considered in the determination of attenuation performance. Nonetheless, taking only certain driving dissipation factors while putting little attention to other significant factors would affect the rate of dissipation. For instance, mangrove age will likewise result in different dissipation rates depending on the maturity of the trees, although the high width of mangroves has been considered. In other words, this means that great width alone could not contribute to high dissipation if young mangroves were assessed.

Therefore, all affecting factors including mangrove structures, wave effects, and hydrodynamics should be incorporated in future studies on mangrove-induced wave dissipation. Bao’s formulation has it all by incorporating all influencing factors in his formula; but, the limitation is that the formulation may overgeneralize among the species of mangroves as the developed equation fits all dominant species (e.g., *Rhizophora mucronata*, *Sonneratia caseolaris*, *Sonneratia griffithii*, *Aegiceras corniculatum*, *Avicennia marina*, and *Kandelia candel*). As such, this should have also been taken into account because different species act differently with the hydrodynamics of waves as they possess different structural characteristics. Thus, it is recommended that future studies segregate the analysis of different species apart from considering all affecting factors in wave attenuation analysis. A new numerical formula might also be produced for wave attenuation determination, but in a detailed categorization according to mangrove species.

**6. Effectiveness of Mangroves in Wave Dissipation**

Mangrove effectiveness varies depending on the conditions of vegetation and hydraulic parameters. As studies have experimented on various vegetation parameters and hydraulic conditions, the rate of wave reduction varies as well. Table 1 shows the reduction rate of several mangrove species tested in different vegetation widths and densities. These researches were carried out in variety wave conditions, consisting of normal wind-induced wave, cyclone-induced wave and tsunami wave. Mangroves shielding function from tsunami wave is a debatable issue. Mangroves is claimed incapable for tsunami protection where in some cases, mangroves get uprooted [69] and reduced their protection ability [70] due to the great energy and massive magnitude of tsunami which eventually become land debris that intruded into the land. While mangroves might not totally deplete the disastrous effect of tsunami, but the damages are lessened [36].

Thus, accounting the various parameters taken, the reduction performance in the following cases might not be comparable among cases, but the dissipation function on several wave conditions is still proven.
Table 1. Dissipation Effectiveness of Different Mangrove Species and Characteristics

<table>
<thead>
<tr>
<th>Species</th>
<th>Mangrove characteristics</th>
<th>Wave Reduction Rate, %</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Kandelia candel</em></td>
<td>Width, m: 100 Density, tree/m²: Not provided</td>
<td>20</td>
<td>Mazda et al. (1997) [71]</td>
</tr>
<tr>
<td><em>Avicennia</em></td>
<td>Width, m: 3, 5, 10, 20, 50 Density, tree/m²: Not provided</td>
<td>60 - 98</td>
<td>Herison et al. (2017) [38]</td>
</tr>
<tr>
<td><em>Sonneratia</em></td>
<td>Width, m: 100 Density, tree/m²: 0.08</td>
<td>50</td>
<td>Mazda et al. (2006) [28]</td>
</tr>
<tr>
<td><em>Rhizophora</em></td>
<td>Width, m: 400 Density, tree/m²: 0.2</td>
<td>30</td>
<td>Yanagisawa et al. (2009) [69]</td>
</tr>
<tr>
<td><em>Rhizophora</em></td>
<td>Width, m: 200 Density, tree/m²: 0.11 (Sparse) 0.16 (Medium) 0.22 (Dense) 0.36 (Super Dense)</td>
<td>77 86 88 91</td>
<td>Hashim and Khairuddin (2014) [72]</td>
</tr>
<tr>
<td><em>Rhizophora</em></td>
<td>Width, m: 50 Density, tree/m²: 11 (Sparse) 16 (Medium) 22 (Dense)</td>
<td>65 74 81</td>
<td>Hashim and Catherine (2013) [36]</td>
</tr>
<tr>
<td><em>Rhizophora</em> model</td>
<td>Width, m: 300 Density, tree/m²: 0.5 – 1.7</td>
<td>60</td>
<td>Narayan et al. (2011) [73]</td>
</tr>
<tr>
<td>Coastal tree model</td>
<td>Width, m: 100 Density, tree/m²: 0.3</td>
<td>50</td>
<td>Mazda et al. (2006) [28]</td>
</tr>
<tr>
<td>Mangrove model</td>
<td>Width, m: Not provided Density, tree/m²: 0.175</td>
<td>49 - 55</td>
<td>Samiksha et al. (2019) [74]</td>
</tr>
</tbody>
</table>

Based on Table 1, it can be summarized that almost all studies did not consider bottom friction calculation, except for Yanagisawa et al. (2009) and Samiksha et al. (2019) [69, 74]. As previously explained, bottom friction is the driving factor influencing the wave reduction, along with the vegetation drag force. Field experiments commonly neglect the individual effect from the bottom friction and rather assume the friction, likewise, as the vegetation drag force. This may result in value overestimation and contribute to some errors. However, Mazda et al. (2006) justified that the bottom friction is negligible only if the water depth is higher, to which the bottom friction appears to be insignificant in reducing the wave height [28]. More accurate data can probably be produced from numerical modeling where several models can include and simulate the bottom friction coefficient in the analysis.

7. Mangroves Rehabilitation and Conservation Efforts in Malaysia

Recognition of the vital role of mangroves as a natural wave barrier has raised awareness on the importance of mangrove conservation and rehabilitation. Besides being destroyed due to climatic changes [75, 76] and natural hazards (e.g., tsunami, cyclone, and erosion [77, 78]), land development has also resulted in mangrove losses. Clearcutting to make room for human activities such as aquaculture ponding and coastal urbanization [79, 80, 81], as well as land-use changes have unfortunately led to ecosystem alteration that causes the degradation of mangroves. This signifies that sustainable mangrove management, conservation, and rehabilitation are crucial for maintaining the effective defense mechanism of mangroves.

The establishment of protected areas in undisturbed regions is the most popular strategy for conserving mangrove ecosystems. Wildlife sanctuaries, national parks, and nature reserves are among the common initiatives [82]. The latest statistics by the Forestry Department of Peninsular Malaysia showed that 90,000 hectares of mangroves in Peninsular Malaysia are classified as Permanent Forest Reserve [83]. By the year 2018 in Sarawak, 11,084 hectares and 12,950 hectares of mangroves have been gazetted as Permanent Forest Estate and Totally Protected Area, respectively [84]. Meanwhile, Mangrove Forest Reserve in Sabah covers approximately 234,680.27 hectares as of 2020 [85].

Since 2005, the Tree Planting Program with Mangroves and Other Suitable Species Along National Coastlines has been regarded as the national rehabilitation program in Malaysia [86, 87] with the aim to restore the ecosystem of mangroves. Comp-Pillow and Comp-Matt planting techniques introduced (refer Figure 3) in this project are among the known techniques in the mangrove research and development area [88]. Both were tested in the mangrove restoration area of Sungai Haji Dorani in Kuala Selangor [89]. Back in 2008, another coastal rehabilitation effort was carried out in Sungai Haji Dorani for sediment trapping and stabilization through the rehabilitation of mangroves [90].
Mangrove roots are widely known for their functionality to accumulate sediments and, thus, stabilizing the shoreline [91, 92, 93]. The coastal structure consisting of the detached breakwater was adopted in this project along with the biotechnical approach for mangrove planting to aid in suitable site conditions for mangroves to establish, grow, and prevent from being washed away by strong waves. Monitoring revealed that 30% of the planted saplings survived after eight months, indicating moderate success. This also means that more than half the mortality rate of saplings was recorded in the project.

On the west coast of Peninsular Malaysia, the rehabilitation efforts are mostly significant, especially after the Indian Ocean Tsunami struck in December 2004. Based on an interview with the coastal communities near Kuala Teriang, Langkawi, mangrove replantation has been implemented at the site along with the discovery of some bamboos expected to be used as techniques during the replantation. This is further proven when a study claimed that the replantation in Kuala Teriang to Sungai Melaka was among the successful efforts [94]. Geotubes of 100 m long were laid in the front beach area for coastal protection measures. In addition, replantation in Lekir, Perak was claimed to have failed even after several attempts have been done.

Sabah with the largest coverage of mangroves in Malaysia [82, 95] was optimistic with its conservation and restoration efforts to date [96]. The enforcement of Forest Enactment 1968 under the state legislature has assured the conservation status of the mangroves, where harvesting for domestic use is only allowed on a small scale. An area of 738 hectares has been rehabilitated throughout four years since 2006. Subsequently, from 2011 to 2014, the Sabah Forestry Department initiated a collaboration with the International Society for Mangrove Ecosystems to enhance the mangrove rehabilitation effort. A total of 1,396.4 ha of mangrove degraded areas have been restored by the end year of 2020 [85].

Currently, the Malaysian Government channels specific allocation in Budget 2021 to support mangrove replantation in Tanjung Piai, Johor, and Kuala Sepetang, Perak, as part of natural resources and biodiversity preservation effort. The Government had also allocated approximately RM48 million for mangrove rehabilitation under the 9th Malaysia Plan, with RM8 million for research and development areas [79, 97]. Nevertheless, one of the common challenges encountered in the rehabilitation project would be the funding issue [85, 96]. The allocation was often inadequate to allow for more sustainable efforts to be performed in the country.

In 2014, the Sabah Forestry Department had suggested an additional allocation for the mangrove project, yet this remains insufficient in 2020 and eventually restricts the scope of its rehabilitation effort. Prior to the national rehabilitation project as stated previously, the State Governments have added their own budget to run the national project. This explains that, instead of the Federal Government alone, the State Government should be more considerable to support similar biodiversity projects through some allocation in the state budget.

After all, every effort from individuals to the government sectors and everyone in between including the non-governmental organizations (NGOs), institutions of higher learning, related agencies, stakeholders, and local communities matters in the conservation and rehabilitation efforts from national to small scale projects. A study by Martinez-Espinosa (2020) by interviewing the local communities near the Matang Mangrove Forest Reserve (MMFR) revealed that the surrounding communities are willing to show their participation in the management and decision-making process of the current management [98].

Public participation and community involvement are also among the key components influencing these efforts. The local community’s participation would instill not only awareness but also ownership towards the mangroves. Aside from that, a profound understanding of forestry, ecological engineering, and coastal engineering must be incorporated for the sustainable and proper planning of mangrove conservation and rehabilitation purposes. This includes the consideration of site-species suitability, planting techniques, environmental aspects of soil and water pH, salinity, hydrology, and wave energy. Thus, better protection to safeguard the coastline can be served with not only extensive
conserved and rehabilitated mangroves sprawl, but also the promising mangrove structures that can withstand severe waves and wind attacks.

8. Implementations of Mangroves as Coastal Bio-shield

The rehabilitation and conservation efforts implemented by several countries have signified mangroves as an important element in protecting their coastline and served as a coastal bio-shield. These countries are including Sri Lanka, Philippines, Gulf Coast of South Florida, and Caribbean Nations, to name a few. The conservation and restoration efforts were made significant especially after evidently benefitted as protection during tsunami, cyclone, and typhoon events.

Mangrove’s coverage in Philippines has been degraded to make way for aquaculture activities such as fish and prawn ponding which were increasing. While numerous replantation efforts with huge allocations were implemented, the mortality rates turned out to be higher, with only 10% to 20% rates of newly planted mangroves survived [99]. Two main factors were analyzed concerning the poor survival which are including wrong species matched with site unsuitability. Despite planting according to their ecology, *Rhizophora* species were chosen instead of the natural colonizer in the sandy substrate coastline area, *Avicennia* and *Sonneratia* species. This preference was rather preferred since *Rhizophora* species are having large propagule which would not have to undergo intensive nursery period due to the smaller seedlings of the other two species. Moreover, the occurrence of *Rhizophora* species is commonly in the sheltered area, which explains the high mortality when planted in the fringing coastal area that is most suitable for *Avicennia* and *Sonneratia*. Figure 4 indicates *Avicennia marina* that colonizes naturally in the respective coastal area versus the favorable species of *Rhizophora* that suffered low survival rates.

![Figure 4. Colonization of a) Avicennia marina species, b) Rhizophora species at the similar habitat][99]

More recent, the protective role of mangroves has brought more attention when the country was hit by Super Typhoon Haiyan in 2013. The disaster was claimed as the deadliest in Philippines [100] and has resulted an estimated death toll of 6,293 people with 28,689 and 1,061 injured and missing, respectively as recorded on 3 April 2014 [101]. The severe storm surges and strong wind caused great losses in lives, property, and livelihood in several islands in the Visayas region [21]. This region, as claimed by [99] was the most vulnerable to typhoon events compared to the bigger islands of Luzon and Mindanao, thereby mangroves replantation was implemented for their buffering function. Aside that, another success replantation was reported in Kalibo Island which supported shoreline stabilization and created protecting zone from typhoon. Despite being low-funded project, Kalibo demonstrated high survival rate and revealed that regular maintenance is the key. As of 2021, the government implemented the Enhance National Greening Program as an extension to National Greening Program in 2011-2016, an initiative to grow 1.5 billion trees in 1.5-million-hectare land for restoration of degraded forest [102].

![Figure 5. Level of protection as indicated by pink-red shades in coastal areas of (a) Jost Van Dyke, (b) Sea Cows Bay, and (c) East End][103]
British Virgin Island, Caribbean benefitted the protective nature of mangrove ecosystem in reducing flood risk especially in three coastal areas of Jost Van Dyke, Sea Cows Bay, and East End [103]. The prediction from their vulnerability model shown that the flood risk can be diminished up to 475m inland even with a small-scale mangroves restoration. They projected a suitable area of 2.8 km² for red mangrove replantation within the three areas which can serve protection from flooding up to 200 m inland at Great Harbour and White Bay, Jost Van Dyke, 300 m inland at Sea Cows Bay, and 475 m inland at the East End. As forecasted, at least 167 buildings in Jost Van Dyke, 285 buildings in Sea Cows Bay, and 268 buildings at East End will receive protection, including the schools, clinic, worship places etc. They also suggested that species - site suitability and effective methods are important to be accounted in any replantation efforts to be successful. Figure 5 depicts the flood protection that may be served by the restoration of red mangroves at the three identified areas.

Mangroves were overexploited for utilization of wood products in Kenya [104]. This has led to mangroves losses aside from other factors such as oil pollution, climate change and salt extraction. The poor cutting planning in mangrove management made the degradation worsen. However, recognition of many other good benefits from mangroves ecosystem had become a turning point for restoration effort. Kenya Marine and Fisheries Research Institute (KMFRI) was pioneering the effort in 1991 and up to 2007, more than a million trees have been replanted with survival rates ranging from 10% to 70% depending on the plantation areas [105]. However, main issue arose in mangrove management whereby there was shortage in basic information and data for the development of inclusive management plan as well as lacking participation from the community.

In Bangladesh, they first implemented the replantation efforts in 1966 [106]. Approximately 60 km mangroves have been planted in the frontal area of their low-lying land by 2013. Sonneratia apetala, among other mangroves planted species, was the top successful in the replantation [107]. Sonneratia apetala created maximum friction and obstruction to the water flow. While this species appeared as the most outstanding in the attenuation performance from storm surges, the Sonneratia planted area were inclined to pest attacks. Hence, multispecies plantation is recommended where the potential species that can colonize in the same muddy substrate zonation would be Avicennia officinalis and Bruguiera gymnorrhiza. They emphasized that for an optimum protection, mangroves should be implemented alongside with other engineering hard structures. A similar claim was also made by [108] which explained construction and maintenance cost of the hard structures can be reduced due to a lower height of structure design.

Thailand encountered massive degradation of mangroves between 1975 and 1996 due to conversion to shrimp ponding. Initially, the areas shown in Figure 6 were all mangroves. Nevertheless, the mangroves were cleared to allow for aquaculture activities which they left only a few lines of mangroves in the frontal areas for protection purpose [29]. Unfortunately, these small coverages of mangroves were unable to withstand the severe wave actions and thus, resulting to mangroves mortality and loss of protection line. This scenario made the coastal communities realized that rehabilitation is required for secure protection. The replantation was however reported high in mortality rates because of strong waves and pest attacks. Thereby, the incorporated various structures such as rock revetments, bamboo breakwaters, concrete-pile breakwaters, and sand sausages or geo-tubes to enhance the growth rates yet, few drawbacks were discovered, e.g., expensive, difficult installation, low materials durability, short lifetime durations and even less effective in dissipating waves.

![Retracted coastline after frontier mangroves destroyed](image)

Figure 6. Conversion of previously planted mangroves area into shrimp ponding [29]
9. Conclusions

Protection against coastal hazards has been identified as an important service offered by mangrove ecosystems. Mangroves demonstrate an impressive resistant towards the incoming severe wave. Their developed and dense structures mitigate the forceful impacts and reduced the wave height and energy. This paper highlights that the performance and effectiveness of mangroves in wave dissipation is relying on various governing factors including mangroves parameters such as width, density, species, age, and hydraulics factors such as water depth, bathymetry, and incident wave height. After reviewing previous research, the following recommendations for future research are proposed:

- Regardless of numerous field assessment, laboratory experiments and numerical modeling have been carried out in proving the dissipation capacity of mangroves, but the focus is commonly concentrated on few influencing factors of mangroves parameters only. While all parameters are important, future study incorporating all governing dissipation factors should be developed where the specific analysis for each species of mangroves need to be considered. This could be possible with the integration of numerical modeling.

- Despite the evidence that support every study approach, there is still a need to validate the hypothesis in different locations and scenarios so that strong and holistic conclusion can be drawn. Different settings may have different affect to the attenuation process, e.g., in terms of bathymetry, wave conditions etc. More comprehensive findings across variety topographies and scenarios may reduce uncertainty.

- The extent of protection of the rehabilitated mangroves is still uncertain and has not been fully addressed, hence their efficacy should be studied to guarantee sufficient protection by mangroves.

In addition to that, previous studies on mangroves protection role suggested the idea of proper coastal management, maintenance and administration are required in conserving and restoring mangroves ecosystem for long term protection security by this vegetation towards the coastline. Acknowledging the possibility of frequent and increasing coastal resilience to future natural disasters, therefore effective conservation and rehabilitation are a pressing concern.

10. Declarations

10.1. Author Contributions

Conceptualization, T.H.; writing—original draft preparation, K.E.A.; writing—review and editing, K.E.A.; visualization, K.E.A.; supervision, T.H.H., A.M.; funding acquisition, H.A.M., T.H. All authors have read and agreed to the published version of the manuscript.

10.2. Data Availability Statement

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

10.3. Funding

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10.5. Conflicts of Interest

The authors declare no conflict of interest.

11. References


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