Research Article

Damage Modelling of Compressed Earth Blocks Stabilised with Cement

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This work aims at studying the mechanical behaviour of compressed Earth blocks (CEBs) and compressed stabilised Earth blocks (CSEBs) with 4% and 8% in weight cement stabilisation. A numerical simulation method based on the nonlinear behaviour law coupling isotropic elasticity damage is proposed to predict the mechanical behaviour of brittle and quasibrittle materials at simple compressive loading conditions. This model relies on the free energy of the material to generate the damage-dependent stress expression in order to bring it closer to the experimental findings. Tests on the geotechnical properties of the three soil samples (MAI, BAM, and GAD) collected in Ngaoundere city, the chief town of the Adamawa region of Cameroon, were carried out. Furthermore, simple compression tests were carried out on samples of dimensions $4 \times 4 \times 4$ cm$^3$ after 28 days of drying. By comparing the experimental and numerical results used, we could notice that the average compressive stresses of CEBs are approximately 4.13 MPa and 4.16 MPa, and the average deformation limits are 0.0068 and 0.0069; concerning the average Young’s moduli, they are about 842.30 MPa and 789.88 MPa, and for 4% cement, we obtained an average compressive strain of about 4.23 MPa and 4.28 MPa, average deformation limit 0.0072 and 0.0075, and Young’s moduli give us 719.16 MPa and 714.06 MPa. At 8% cement dosage, we obtained average compressive stresses of about 5.01 MPa and 5.20 MPa, average deformation limit of 0.0073 and 0.0074, and Young’s moduli give us 866.43 MPa and 872.56 MPa.

1. Introduction

Compressed Earth block construction has had a renewed interest in development policies in recent decades. Several studies have been carried out on compressed stabilised Earth blocks (CSEBs) [1–3]. Today, more than one-third of our planet’s inhabitants still live in earth-based habitats. Besides, Earth has many advantages as far as environmental, social, cultural, and economic aspects are concerned [4]. However, the use of raw Earth as a building material for adobe bricks presents significant limitations, such as the high absorption rate due to the relatively high porosity, the formation of drawback during drying, and a low resistance to humidity. When compressed and stabilised under optimal conditions, however, a compressed stabilised Earth block (CSEBs) with good insulation performance and compressive strength is obtained [5]. Therefore, the process of stabilising is a way to improve the mechanical characteristics of Earth blocks [6]. One of the concepts of brittle damage mechanics was initially established by Kachanov [7] and then developed by other authors such as Lemaitre and Chaboche [8] and Krajcinovic [9]. In this study, we performed experimental tests of simple compression with samples of dimensions $4 \times 4 \times 4$ cm$^3$ after 28 days of drying. The numerical method used is the nonlinear least-squares method which allows determining the relative differences between the types of results by minimizing the errors. The best retained for the
2.1. Geotechnical Identification of Soils. Determining the geotechnical properties of the soil will help overcome and predict the behaviour of the soil samples. The tests described below were carried out on our three soil samples at the Local Materials Promotion Authority (MIPROMALO) laboratory in Yaoundé. We successively carried out particle size analysis by dry method after washing (grains greater than 80 μm), particle size analysis by sedimentometry (grains less than 80 μm), methylene blue tests, and we determined the Atterberg limits (liquidity limits, plasticity limits, and plasticity index).

2.2. Location of the Study and Sampling Areas. The soil samples were collected in the Adamawa region (Ngaoundere chief town), located in the northern part of Cameroon, in the VINA division and more precisely in Ngaoundere I (Bamyanga BAM), Ngaoundere II (Gadamabanga GAD), and Ngaoundere III (Maiborno MAI) subdivisions. Soil samples were collected at 60 cm depth in these three neighbourhoods (Figure 1).

2.3. Mechanical Compression Tests

2.3.1. Production of Test Samples for CEBs and CSEBs. We made $4 \times 4 \times 4$ cm$^3$ compressed soil brick samples of CEBs and CSEBs with different percentages of CPJ 35 cement stabilisation at 4% and 8%, according to the following procedure as shown in Figure 2.

(i) Dig the soil to a depth of 60 cm and collect the soil
(ii) Weigh with a scale the masses of soil sufficient for making $4 \times 4 \times 4$ cm$^3$ samples
(iii) Weigh the quantity of CPJ 35 cement at 4% and 8% of the soil mass and then proceed with dry mixing
(iv) Hydrate the whole (Earth + cement) with about 12% of water
(v) Compress the sample with a compaction pressure of 3 MPa

(vi) Dry the samples for 28 days

2.3.2. Simple Compression. Compression tests were carried out using an IMPACT electric press on CEBs and CSEBs samples of size $4 \times 4 \times 4$ cm$^3$ after 28 days of drying (Figure 3). The image on the left shows the nondamaged sample and on the right the damaged one after the crack.

2.4. Damage Modelling

2.4.1. Crack Propagation. Let us examine a cubic sample with a total surface area. When this sample is subjected to a simple compressive strength, microcracks (defects) appear upon reaching a certain value of the load and spread progressively over the surface under consideration (Figure 4).

2.4.2. Formulation

(1) General Form of the Free Energy. The starting point of our work is based on the free energy of Pham’s model [11, 12]. The general form of Pham’s free energy allows describing the behaviour of a cracked material (concrete) taking into account the friction between cracked lips at a macroscopic scale. We have

$$
\rho \Psi = W_1(\varepsilon, D) + W_2(\varepsilon D, \varepsilon^2) + W_s,
$$

$$
\rho \Psi = \frac{1}{2} E (1 - D) \varepsilon^2 + \frac{1}{2} E_g (D - \varepsilon^2)^2 + W_s,
$$

where $W_1 = $ purely elastic free energy; $W_2 = $ anelastic free energy; $W_s = $ blocked energy density; $D = $ damage; $\rho$
density of the material; $\Psi$ = free energy; $\varepsilon$ = deformation; $\varepsilon^\pi$ = deformation related to the friction of the lips of a macrocrack.

(2) Free Energy of the Material. We are suggesting a separation of behaviour between the matrix which resists damage and the cracks which propagate in the material, by considering the deformation linked to the friction of the lips of a zero macrocrack ($\varepsilon^\pi = 0$) due to the separation of the lips of the crack and the density of the blocked energy $W_\Sigma = 0$ because the material is taken in the mesoscopic state (the crack is continuous). The form of the free energy therefore becomes

$$\rho\Psi = W_m(\varepsilon, D) + W_f(\varepsilon, D),$$

$$\rho\Psi = \frac{1}{2} E (1 - D) \varepsilon^2 + \frac{1}{2} D \varepsilon^2,$$

where $W_m = $ free energy of the matrix, $W_f = $ free energy at the tip of the main crack, and $\rho = $ density of the material.

(3) Expressing Damage. The most common areas where damage occurs are in singularity zones like around the interface between phases or in highly concentrated inclusion zones rich in preexisting defects [13]. The properties of the matrix (its hardness and stiffness) in relation to those of the reinforcement can influence the damage mechanism. According to Babout, decohesion dominates in matrices considered soft, whereas particle failures are the dominant mechanism in rigid matrices [14]. The approach we adopted in this work assumes that the damage is isotropic and $D$ is a scalar evolving in the same direction as the deformations. In the following point, we assume that the strain $\varepsilon$ is in the interval $0 \leq \varepsilon < \varepsilon_R$, where $\varepsilon_R$ is the strain at failure and $s$ is the damage factor [10].

$$D = \left( \frac{\varepsilon}{\varepsilon_R} \right)^s.$$  

(4) Expressing Total Stress. Stress, generally occurring in the vicinity of inclusions, can become high enough to break the inclusion by cleavage or to separate the interface between the inclusion and the matrix. Total strain is obtained by deriving the free energy (6) with respect to the stress as

$$\sigma = E (1 - D) \varepsilon + D \varepsilon.$$  

Given the low strains in the closed cracks (microcracks not leading to failure and voids) of the material $\sigma_0$ and to separate the different behaviours

$$\sigma = \sigma_0 + E (1 - D) \varepsilon + D \varepsilon.$$  

The relation (7) can be put into the following form:

$$\sigma = \sigma_0 + \sigma_m + \sigma_f,$$

where $\sigma =$ stress of the material, $\sigma_m =$ matrix stress, $\sigma_f =$ main crack stress leading to failure, and $\sigma_0 =$ microcrack stress of voids (defects).
We obtain the following by substituting the damage expression (5) into (7):

$$\sigma = \sigma_0 + E \varepsilon + \frac{(1 - E)}{\varepsilon R} \varepsilon^{s+1},$$  

(9)

where $s$ is the damage factor of the material. Relation (9) is put into the following polynomial form:

$$\bar{\sigma} = \bar{\sigma}_1 + \bar{\sigma}_2 \varepsilon + \bar{\sigma}_3 \varepsilon^{s+1}.$$  

(10)

The previous approximation model is obtained using the least-squares method; the following simplified matrix form is obtained:

$$
\begin{bmatrix}
1 & \bar{\varepsilon} & \bar{\varepsilon}_{s+1} \\
\bar{\varepsilon} & \bar{\varepsilon}_2 & \bar{\varepsilon}_{s+2} \\
\bar{\varepsilon}_{s+1} & \bar{\varepsilon}_{s+2} & \bar{\varepsilon}_{s+1(s+1)}
\end{bmatrix}
\begin{bmatrix}
\bar{\sigma}_1 \\
\bar{\sigma}_2 \\
\bar{\sigma}_3
\end{bmatrix}
= 
\begin{bmatrix}
\sigma_0 \\
\sigma_1 \\
\sigma_{s+1}
\end{bmatrix}.
$$  

(11)

Solving this system (11) gives us the solution depending on the damage coefficient ($s$). The degree of approximation of the obtained curve is based on the estimated experimental points of the findings.

The maximum compressive strength $R_c$ and the ultimate strain $\varepsilon_{\text{lim}}$ are deduced from this solution of the system.

$$R_c = \sigma(\varepsilon_{\text{lim}}),$$  

$$\varepsilon_{\text{lim}} = \varepsilon_R \left( \frac{-E}{(s+1)(1-E)} \right)^{1/\varepsilon}.$$  

(12)

3. Results and Discussion

3.1. Laboratory Results. The particle size distribution of the three soil samples in Figure 6 shows that the highest proportions of grains in GAD samples are sand and silt, so we have sandy-silt type soils out of the ideal zone whereas sand and clay are the highest proportions of grains in MAI and BAM, so we have sandy-silt-clay in the ideal zone. The liquidity and plastic limits and the plasticity index are summarised in Table 1. For the plasticity indexes (PI) of these samples ranged from 15–40, we have plastic soils. The result of the methylene blue test is presented in Table 1, and the sample with the lowest clay content is that of BAM with 52.50 m$^2$ per 1 g. With regard to the blue activity index, all three samples are between 5 and 13, so we have soils with medium active clay levels. Figures 7–9 show the evolution of the compressive strength as a function of the deformation, as well as the experimental points of the CEBs and CSEBs stabilised with 0%, 4%, and 8% cement. The simple compression strength of CEBs is shown in Figures 7–9, and for 4% cement, we obtained the 4.23 MPa and 4.28 MPa, and at 8% cement dosage, we obtained average simple compression strength of about 5.01 MPa and 5.20 MPa.

3.2. Discussion

3.2.1. Results of Geotechnical Soil Identification Tests. The results of geotechnical soil identification tests of the complete particle size distribution (by the dry method after washing and sedimentometry), the Atterberg limits (liquidity limits (LL), plasticity limits (LP), and plasticity index (IP)), and the methylene blue tests (VBS) are presented in Table 1 [15].

3.2.2. Complete Particle Size Distribution. The three soil samples analysed are recommended for the manufacturing of CEBs according to the NF P94-056 [16] and NF P94-057
standards which define the range of the recommended spindle (Figure 6).

3.2.3. Experimental and Numerical Results. Experimental and numerical findings in simple compression are shown, respectively, in Figures 7 to 9 for MAI, BAM, and GAD $4 \times 4 \times 4$ cm$^3$ samples and for three cement dosage rates (0%, 4%, and 8%). The changes in strains related to the deformations of the numerical part took into account the damage coefficient (s) (Figure 7), of which influence will be illustrated in the following.

(1) Experimental and Numerical Results of MAI’s CEBs and CSEBs. The changes in strains related to the deformations of MAI’s CEBs and CSEBs are shown in Table 2.

MAI findings show an increase in compressive strength and Young’s modulus as the cement content increases too. The average margin of error is quite low at about 1% for compressive stress and 1% for Young’s modulus, but the average margin of error for ultimate strain is 6%. This

| Geotechnical identification tests | Particle size distribution (proportions of particles) | Atterberg limits | Methylene blue value |
|----------------------------------|-----------------------------------------------|-----------------|---------------------|
| Sampling sites                  | Gravels | Sand | Silt | Clay | LL (%) | PL (%) | PI (%) | VBS (g/100 g) | $S_p$ (m$^3$/g) | $A_{CB}$ |
| MAI                             | 1.5     | 53.3 | 28.2 | 17   | 44.30  | 26.10  | 18.60  | 4.33       | 90.93      | 11.04   |
| BAM                             | 14      | 48.2 | 14.3 | 23.5 | 43.59  | 4.18   | 19.40  | 2.50       | 52.50      | 07.09   |
| GAD                             | 10.5    | 55.5 | 26   | 8    | 44.66  | 24.33  | 20.32  | 3.16       | 66.48      | 10.25   |

Table 1: Results of the geotechnical identification tests.

![Figure 7: Simple compression stress based on the strain of MAI (0%, 4%, and 8%).](image-url)
Table 2: Mechanical characteristics of MAI’s CEBs and CSEBs.

| Dosage (Cement (%)) | Maximum strains $R_c$ (MPa) | Deformation limits $\varepsilon_{\text{lim}}$ | Young’s modulus $E$ (MPa) |
|---------------------|-----------------------------|---------------------------------------------|--------------------------|
|                     | Exp | Num | Error | Exp | Num | Error | Exp | Num | Error |
| 0                   | 4.20 | 4.26 | +1%  | 0.0060 | 0.0067 | +12%  | 840.30 | 836.24 | −1%  |
| 4                   | 4.23 | 4.18 | −1%  | 0.0071 | 0.0075 | +6%   | 735.10 | 730.32 | −1%  |
| 8                   | 5.12 | 5.16 | +1%  | 0.0074 | 0.0073 | −2%   | 870.98 | 878.34 | +1%  |

Figure 8: Simple compression stress based on the strain of BAM (0%, 4%, and 8%).

(2) Experimental and Numerical Results of BAM’s CEBs and CSEBs. The changes in strains related to the deformations of BAM’s CEBs and CSEBs are shown in Table 3.

BAM findings show an increase in compressive strength and Young’s modulus as the cement content increases, and the average margin of error is quite low at about 1% for compressive strength and 1% for Young’s modulus, but the average margin of error for ultimate strain is 5.33%. This accuracy shows a consistency between experimental and numerical findings, as shown in Figure 8.

(3) Experimental and Numerical Results of GAD’s CEBs and CSEBs. The changes in strains related to the deformations of GAD’s CEBs and CSEBs are shown in Table 4.

GAD findings show an increase in the compressive strength and Young’s modulus as the cement content increases, and the average margin of error is quite low at about 1.33% for compressive strength and 1% for Young’s
Table 3: Mechanical characteristics of BAM’s CEBs and CSEBs.

| Dosage (Cement (%)) | Maximum strains $R_c$ (MPa) | Deformation limits $\varepsilon_{lim}$ | Young’s modulus $E$ (MPa) |
|--------------------|------------------------------|-------------------------------------|------------------------|
|                    | Exp  | Num  | Error | Exp  | Num  | Error | Exp  | Num  | Error |
| 0                  | 4.20 | 4.22 | +1%   | 0.0071 | 0.0072 | +2%   | 788.02 | 793.82 | +1%   |
| 4                  | 4.55 | 4.51 | −1%   | 0.0071 | 0.0074 | +4%   | 752.86 | 744.84 | −1%   |
| 8                  | 5.40 | 5.48 | +1%   | 0.0071 | 0.0078 | +10%  | 872.99 | 876.09 | +1%   |

Figure 9: Simple compression stress based on the strain of GAD (0%, 4%, and 8%).

Table 4: Mechanical characteristics of GAD’s CEBs and CSEBs.

| Dosage (Cement (%)) | Maximum strains $R_c$ (MPa) | Deformation limits $\varepsilon_{lim}$ | Young’s modulus $E$ (MPa) |
|--------------------|------------------------------|-------------------------------------|------------------------|
|                    | Exp  | Num  | Error | Exp  | Num  | Error | Exp  | Num  | Error |
| 0                  | 4.01 | 4.00 | −1%   | 0.0073 | 0.0068 | −7%   | 730.78 | 739.59 | +1%   |
| 4                  | 4.16 | 4.15 | −1%   | 0.0075 | 0.0077 | +3%   | 669.53 | 667.03 | −1%   |
| 8                  | 5.01 | 5.07 | +2%   | 0.0075 | 0.0073 | −3%   | 855.34 | 863.54 | +1%   |
3.2.4. Effect of Cement Stabilisation. The experimental findings of the compressive strength are in agreement with the studies of Bahar et al. Bahar et al. showed that the compression and tensile strength by splitting increases with increasing cement content [18].

Our experimental findings show that the simple compressive strength values for 4% CEBs and CSEBs are higher than those of Bahar and Tran [18, 19]. However, at 8%, our values are lower than those of Tran but higher than those of Bahar. Bahar et al. showed that the compression and tensile strength by splitting increases with increasing cement content [18].

The cement content in CSEBs can be increased to the desired value for a given strength. The findings show that our values for CEBs and CSEBs simple compressive strength at 8% in weight of cement are higher than those of Dao et al. [20]. However, at 4%, our findings are lower than those obtained by the latter. The main effect of cement stabilisation is the water insensitivity of the material. The compressive strength remains very dependent on the dosage, and 8% in the weight of cement is often the economically acceptable upper limit [20].

Akpokodje examined the effect on the strength of soil with different cement contents. He found that the compressive strength increases with the cement content.

Figure 10: Young’s modulus compared to the damage coefficient of CEBs and CSEBs.
according to a linear function [21]. Adding cement to a soil significantly increases its compressive strength [21, 22]. This finding is consistent with our experimental findings.

A comparative study between the experimental and numerical findings of the CEBs and CSEBs made it possible to determine several parameters, in particular the compressive strength ($R_c$), the deformation limit ($\varepsilon_{\text{lim}}$), and the Young’s modulus ($E$). This study was carried out by Ntamack et al. [23]. Based on all these findings, cement appears to be the essential element in the optimisation of formulations and, consequently, contributes to improving the mechanical performance of mud bricks. Doat asserts that the main effect of cement stabilisation is the water insensitivity of the material [24].

3.2.5. Influence of the Damage Coefficient on Young’s Modulus. The analytical study shows the influence of the damage coefficient ($s$) on the Young’s modulus ($E$). This is illustrated in Figure 10 of the previously obtained numerical findings.

The numerical findings show a progressive decrease in Young’s modulus ($E$) as the damage coefficient increases for all the CEBs and CSEBs samples. It should be noted that there is a difference in the behaviour of CSEBs, and at 4% cement, there is a drop in the Young’s modulus values compared to CEBs. However, the values of Young’s modulus increase with 8% cement compared to CEBs.

3.2.6. Influence of the Damage Coefficient on the Simple Compression Stress. The analytical study shows the influence of the damage coefficient ($s$) on compression stress ($R_c$). This is illustrated in Figure 11 of the previously obtained numerical findings.

The numerical findings show a growth of the simple compressive strength when the damage coefficient increases, at $s = 4$. We observe that the numerical findings are very
accurate compared to those obtained experimentally. The superposition of the curves between CEBs and CSEBs at 4% shows a weak growth between CEBs and CEBs stabilised at 4%, but at 8% CSEBs, a clear difference is observed as compared to the others.

4. Conclusion

This work proposes a study of the damage modelling of cement-stabilised Earth blocks and Earth blocks. The numerical approach of solving by the least-squares method provided us with interesting results, which allows us to state that this method is effective in predicting the damage of CEBs and CSEBs. This study reveals that cement gives some gain in compressive strength of stabilised Earth bricks depending on the rate of stabilizer used, which is of paramount importance in the field of construction from the ecological and economical point of view. A comparative study between the experimental and numerical methods finding of the CEBs and CSEBs with 4% and 8% in weight cement stabilisation made it possible to determine several parameters, in particular the compressive strength, the deformation limit, and the Young’s modulus. We have analysed the compressive stress values, the deformation limit, and Young’s modulus of the CEBs and CSEBs obtained accuracy which show a consistency between the experimental and numerical results. The numerical results show the influence of the damage coefficient on the stress and on Young’s modulus. We can therefore state that it is possible to use this model to predict the damageable behaviour of compressed Earth blocks (CEBs) and cement-stabilised compressed Earth blocks (CSEBs).

Data Availability

The data used to obtain the results of this study are included in the article.

Conflicts of Interest

The authors declare that they have no potential conflicts of interest with respect to the research, authorship, and/or of this article.

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