Research Article

Analysis of Uniaxial Compression Mechanical Properties of Rubber Powder Recycled Coarse Aggregate Concrete Based on Strain Energy Theory

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Received 6 June 2022; Revised 4 August 2022; Accepted 6 August 2022; Published 30 August 2022

1. Introduction

In recent years, the amount of concrete used worldwide has exceeded 4 billion cubic metres per year, and the amount of sand and gravel materials used has exceeded 6 billion cubic metres per year and can even reach 8 billion cubic metres per year [1]. Although the world’s economic boom is driven by the extraction and exploitation of natural resources, the economic development and the improvement of people’s material standards have brought with them some environmental problems. For example, the extraction and processing of natural resources requires a large amount of energy [2]. The expansion of the construction industry requires energy consumption, which generates large amounts of carbon dioxide and causes serious carbon pollution, which has consequently become a major global pollution problem [3, 4]. The development of the construction industry inevitably causes the demolition of a large number of old buildings, which generates a large amount of solid waste. With the acceleration of urbanization in China, it generates about 30% of the world’s municipal solid waste, 40% of which is construction waste [5, 6]. For recycling organizations, the resource utilization of construction waste is often time-consuming and labour-intensive, while only 20–30% of construction waste is recycled globally due to insufficient use of technical tools [7–9]. Currently, the dumping and land-filling of waste concrete causes soil, air, and water pollution [10].

The approach of crushing waste concrete into recycled aggregates to replace natural gravel is an effective method of...
recycling waste concrete. It not only relieves the pressure of natural resource extraction and reduces energy consumption and solves the pollution problem caused by waste concrete but also enables the sustainable development of waste concrete [11, 12]. Recently, a great deal of research has been carried out by scholars on the performance of recycled aggregate concrete. Some scholars use the recycled coarse aggregate in reinforced concrete beams to carry out tests on the flexural performance of the beams. It was found that the shear bearing capacity of recycled coarse aggregate concrete beams is low compared to that of ordinary concrete beams, and the crack width of recycled coarse aggregate reinforced concrete beams increases about 16% larger than that of ordinary reinforced concrete beams when the recycled coarse aggregate replacement rate reaches 100% [13, 14]. Other scholars have studied recycled aggregates as a substitute for rock aggregates in asphalt concrete and found that recycled aggregates are feasible as replacement aggregates, and the best strength performance of recycled aggregate asphalt concrete can be obtained when recycled aggregates replace natural aggregates according to 40% [15, 16]. The splitting tensile and compressive strengths of recycled aggregates are reduced by 10% and 25%, respectively, compared with natural aggregates [17]. This resulted in the compressive and splitting tensile strengths of recycled aggregate concrete are lower than those of ordinary concrete. Therefore, researchers incorporated industrial fibres to improve the mechanical properties of recycled concrete. For example, the incorporation of basalt fibres improved the cracking resistance of recycled concrete [1]. When glass fibres [18], polypropylene fibres [19], and steel fibres [20] were incorporated into the recycled concrete, the compressive strength, flexural strength, and stiffness of the recycled concrete were improved, respectively.

Some scholars have conducted research on recycled fine aggregates. For example, when the recycled fine aggregate replaces 50% of the natural sand, concrete with a strength of C40 can be configured [21]. It was found that the higher the amount of nano-scale silica, the higher the strength of the cementitious sand by mixing nano-scale silica dioxide into the cement and replacing some of the natural sand with recycled fine aggregate to produce cementitious sand [22]. Some scholars have made recycled concrete powder from waste concrete and mixed it into self-compacting concrete with cement containing nanoscale silica for research, and found that the recycled concrete powder could be used directly without further treatment and that the early hydration rate of the cement was found to be increased [23, 24]. Coarse aggregates used for the test concrete were all studied with recycled coarse aggregates.

At least 20 billion waste rubber tyres will be generated worldwide every year [25]. This treatment method consumes large land resources and also may contaminate soil and groundwater due to the nonbiodegradable nature of rubber thus producing harmful diseases [26–28]. If waste tyre rubber powder is added to concrete as an admixture, not only the certain properties of the concrete can be changed, but also the waste tyre rubber powder can be recycled. Therefore, many researchers have tested the incorporation of waste tyre rubber powder into concrete with satisfactory results. Compared with traditional concrete, rubber concrete is lighter in weight and lower in density, has better impact resistance, and has good thermal insulation properties [29–32]. For these reasons, the study of rubber concrete has become a popular research domain in recent years.

When rubber powder was used to replace natural sand at the proportions of 5%, 10%, 15%, and 20% by quantity and quality, it was found that the higher quantity of the rubber powder replacement, the lower compressive strength of the concrete [33]. After the surface modification of rubber powder particles by silane coupling agents, the compressive and flexural strengths of rubber concrete were still reduced, but the bonding surface of rubber powder and concrete was improved [34]. Scholars have studied the application of waste rubber powder in asphalt concrete. Concrete material (B) containing rubber powder particles was incorporated into asphalt concrete, and when B was incorporated at 20%, the indirect tensile strength and modulus of stiffness were reduced by 50% compared with the ACC test group [35]. In asphalt concrete, the rubber additive increased the softening point of the asphalt cement, reduced the permeability and ductility of the cement, and improved chemical stability and ageing resistance [36, 37].

In terms of concrete durability, scholars have found that the rubber powder concrete has better frost resistance compared with the ordinary concrete in a sulphate environment, and the wear resistance of rubber concrete has been improved [38, 39]. The frost resistance of concrete was improved when rubber powder was used to replace fine aggregates [40]. Rubber powder particles have a tendency to blunt cracks and can increase the ability of the specimen to absorb damage energy and deform with minimal crack width [41]. It has also been shown that rubber concrete has a high resistance to corrosive environments and can be used in acidic environments [42]. In addition, rubber powder concrete has good toughness and ductility [43]. In summary, the use of waste rubber powder can not only utilize the waste but also improve the performance of concrete. In the experiment, waste rubber powder will be mixed in concrete and the amount of rubber powder will be fixed to study the effect of different mesh of rubber powder on the strain energy and microscopic pore structure of recycled concrete.

2. Experimental Procedure

2.1. Materials. Ordinary portland cement with a grade of 42.5 and the density is 3093 kg/m³, which produced by Tangshan Jidong Cement Co., Ltd. Medium-sized river sand, with a fineness modulus of 2.5 and an apparent density of 2623 kg/m³, was used. The coarse aggregates selected recycled coarse aggregates, with a particle size range of 5–20 mm and apparent density of 2483 kg/m³, which were produced by jaw crusher. The crushed value of recycled coarse aggregates was 15.76%. Rubber powder is 20 mesh, 60 mesh, and 100 mesh, which were produced by waste tire rubber powder processing factory in Sichuan province, China. The amount of rubber powder is 3% of cement. FDN-C naphthalene superplasticizer was used as the water-reducing
2.2. Production Process of Recycled Coarse Aggregates. The recycled coarse aggregate (RCA) is generated from the crushing of waste C40 concrete, and the photo of ready-made recycled coarse aggregate is shown in Figure 1. The waste C40 concrete is derived from the same element of the abandoned building structure. The waste concrete was first discarded as the outer surface of the waste building element, which had been carbonized and eroded. It was then discarded from the reinforcement bond, as the deformation of the reinforcement had caused damage to the surrounding concrete and rust, which is considered an impurity, had contaminated the part of the concrete. The remaining waste concrete is first crushed with a sledgehammer to produce small pieces of 6 cubic centimetres, which are then crushed by a jaw crusher to produce a smaller particle, and finally, the crushed material is sieved through a test sieve to produce a coarse recycled aggregate with a particle size of 5–20 mm.

The homogenization process of recycled coarse aggregates is very important. Homogenized recycled aggregates are beneficial in reducing the differences in mechanical properties of the tested concrete. In this test, the recycled coarse aggregates are homogenized and sampled for analysis before the concrete is configured. Khoury et al. [44] provided a method for homogenization and sampling of recycled aggregates and found that this homogenized aggregates were better evaluated. The method was as follows: 1310.5 kg of recycled aggregate was shovelled into the bucket of the bulldozer, which then dumped the aggregate in the bucket on the ground and spread it evenly; this was repeated three times. After that, the aggregate dumped on the ground was spread into a circle as far as possible, and grid lines (spacing 30 cm) were applied on the surface of the paved recycled aggregate, and the recycled aggregate within 12 squares was evenly selected as the samples for the first sampling.

Using the above method and based on the characteristics of the concrete mix for this test, 130 kg of recycled coarse aggregate was considered sufficient for this test. Therefore, a tipping cart was used instead of a bulldozer. Firstly, the recycled coarse aggregate was mixed by concrete mixer and then loaded into the tipping bucket of the tipping cart, and then, the recycled coarse aggregate in the tipping bucket was dumped on the ground and spread evenly; it was repeated 3 times. Then, the aggregate dumped on the ground was paved into a circle whenever possible, grid lines (spacing 10 cm) were applied on the surface of the paved recycled aggregate, and the recycled aggregate within 12 squares was uniformly selected as the first selected sample. At this point, the homogenization of the recycled aggregate and the first sampling was completed. The first sample was sampled using the upper homogenization method and the sampling method, and the recycled aggregate within 6 squares was selected as the second sample. The second sample was sampled using the upper homogenization method and the sampling method, and the recycled aggregate within 3 squares was selected as the third sample. The weight of the third recycled aggregate sample was less, with an average of 0.55 kg per sample. Therefore, the third sampling was used as the final sampling and the samples within the three squares were named as Group A, Group B, and Group C, respectively. The SEM photographs of the recycled aggregates with particle sizes of 5–10 mm, 10–15 mm, and 15–20 mm selected from groups A, B, and C are shown in Figure 2.

From Figure 2, it can be seen that the old mortar fraction of recycled aggregates at all three-grain sizes in groups A, B, and C have different degrees of damage cracks. These damaged cracks may have an effect on energy dissipation. However, the uniformity of these cracks in groups A, B, and C could not be determined by SEM photographs, so a mercury-pressure test was used to investigate the amount of cracks in the old mortar fraction by porosity. One sample was taken from each of groups A, B, and C, old mortar fractions of three particle sizes (5–10 mm, 10–15 mm, and 15–20 mm) of aggregate were extracted and mixed and weighed at 10 ± 0.5 g. Table 1 shows the results of the mercury-pressure test. From Table 1, it can be seen that the difference in porosity between groups A, B, and C is within 2.63% and the difference in total intrusion volume of Mercury is within 3.03%. Since the variations are quite small, it is judged that the fracture contents of the recycled aggregates in groups A, B, and C are more uniform, and thus, it is concluded that the fracture contents of the recycled aggregates for the test are more uniform.

2.3. Experimental Design

2.3.1. Experimental Design of Mechanical Property. The reference concrete group was named as RC, with a design strength grade of C40, which was made entirely from recycled coarse aggregate. The test concrete groups were made entirely from recycled coarse aggregate and mixed with rubber powder, concurrently. The test concrete group mixed with rubber powder was named as RC20, RC60, and RC100, with rubber powder mesh of 20 mesh, 60 mesh, and 100 mesh, respectively. The amount of rubber powder is 3% of cement. The mixture proportion of the control group was designed following JGJ 55–2011 (2011) [45]. The material composition of concrete is provided in Table 2.

In Table 2, the amounts of cement, water, sand, and recycled coarse aggregate in the RC, RC20, RC60, and RC100
Figure 2: SEM images of recycled aggregate. (a) SEM of 5–10 mm RCA of A. (b) SEM of 10–15 mm RCA of A. (c) SEM of 15–20 mm RCA of A. (d) SEM of 5–10 mm RCA of B. (e) SEM of 10–15 mm RCA of B. (f) SEM of 15–20 mm RCA of B. (g) SEM of 5–10 mm RCA of C. (h) SEM of 10–15 mm RCA of C. (i) SEM of 15–20 mm RCA of C.

Table 1: Test results of mercury intrusion porosimetry.

| Group | Sample weight (g) | Total intrusion volume (mL/g) | Average pore diameter (nm) | Porosity (%) |
|-------|-------------------|-----------------------------|---------------------------|--------------|
| A     | 10.14             | 0.0165                      | 24579.3                   | 3.4108       |
| B     | 10.02             | 0.0160                      | 31234.7                   | 3.4531       |
| C     | 10.30             | 0.0163                      | 23341.8                   | 3.3624       |

Table 2: Mix proportion.

| Group | Cement (kg/m³) | Water (kg/m³) | Sand (kg/m³) | Rubber powder mesh | Rubber powder mixing amount (kg/m³) | Recycled coarse aggregate (kg/m³) | Water-reducing agent (kg/m³) |
|-------|----------------|---------------|--------------|--------------------|------------------------------------|----------------------------------|------------------------------|
| RC    | 370            | 166.5         | 731          | —                  | —                                 | 1096                             | 3.70                         |
| RC20  | 370            | 166.5         | 731          | 20                 | 11.1                              | 1096                             | 3.70                         |
| RC60  | 370            | 166.5         | 731          | 60                 | 11.1                              | 1096                             | 3.70                         |
| RC100 | 370            | 166.5         | 731          | 100                | 11.1                              | 1096                             | 3.70                         |
groups are the same: 370 kg/m$^3$, 166.5 kg/m$^3$, 731 kg/m$^3$, and 1096 kg/m$^3$, respectively. The water-cement ratio is 0.45. The amount of water reducing agent is mixed at 1% of the cement mass, that is, 3.70 kg/m$^3$. The rubber powder in RC20 group, RC60 group, and RC100 group is 20 mesh, 60 mesh, and 100 mesh, and the amount of mixing is 3% of cement mass, that is, 11.1 kg kg/m$^3$.

Some researchers have used concrete cubic blocks for testing stress-strain curves. Stress-strain curves of concretes with different size steel aggregates were obtained by uniaxial compression experiment with concrete test cubes [46]. In order to explore the influence of initial defects on the deformation and failure of concrete, the stress-strain curve was obtained by uniaxial compression experiment with concrete test cubes [47]. The block for uniaxial compression test had the dimensions of 100 mm $\times$ 100 mm $\times$ 100 mm, and the electro-hydraulic servo pressure testing machine was used to record 14 days (14 d), 28 days (28 d), and 90 days (90 d) stress-strain data. The block for initial elastic modulus test had the dimensions of 150 mm $\times$ 150 mm $\times$ 300 mm, and the exclusive elastic modulus instrument of the electro-hydraulic servo pressure testing machine was used to test initial elastic modulus. The electro-hydraulic servo pressure testing machine is shown in Figure 3.

2.3.2. Pore Structure Test and Scanning Electron Microscopy Test. In order to study the pore structures of the cut surface of test concrete groups, RapidAir457 stomatal structure analyzer was used for collecting data. The RapidAir457 stomatal structure analyzer is shown in Figure 4. The RapidAir457 system consists of a computer control unit (PC), colour monitors, a camera lens and microscopic objective, and an analysis software.

The steps of making samples are as follows: first, we used a cutting machine to cut the concrete cube test block into a square test piece with a thickness of 15 mm. Then, we cut off the corners of the square lamina sample and made it into a regular octagonal specimen. Furthermore, we polished the surface of the regular octagonal specimen and then smeared the polished sample surface with black ink and waited for it to dry. Finally, the prepared nano-CaCO$_3$ paste was applied to the surface of the sample and gently pressed. Finally, we scraped off the excess paste with a knife and RapidAir457 stomatal structure analyzer was used to test the above samples. In order to further study the microstructure of rubber powder and recycled coarse aggregate in concrete, we used scanning electron microscopy for imaging analysis. The scanning electron microscopy is shown in Figure 5.

3. Results and Analysis

3.1. Stress-Strain Curves at Different Ages. Figure 6 shows the stress-strain curves for each test group at 14, 28, and 90 days of curing age. The two groups with the highest cutline modulus of elasticity at initial loading (strain of $0 \sim 0.2 \times 10^{-3}$) at the age of 14 d were RC (46.417 GPa) and RC100 (53.723 GPa) and the lowest was RC20 (37.948 GPa), but the RC20 group had the highest strain at the peak load. The RC and RC100 groups exhibit low strain and high stiffness brittle characteristics. The RC20 group can mitigate the brittleness by improving its own deformation performance. The peak stresses of the RC20, RC60, and RC100 groups were 44.327 MPa, 40.879 MPa, and 40.375 MPa at the curing age of 28 days. The trend was that the peak stresses decreased gradually with the increase of the mesh of rubber powder. When the curing time was 90 days, the compressive strength of the rubber powder recycled concrete was lower than that of the test group without rubber powder, which indicates that the weakness of rubber powder as a nonconstruction material is prominent at this time, as shown by the
weakening of the strength of the recycled concrete. At a curing age of 90 days, the strain values at which the slope of the strain curve began to decrease significantly in the RC group, RC20 group, RC60 group, and RC100 group were $1.0 \times 10^{-3}$, $0.9 \times 10^{-3}$, $0.8 \times 10^{-3}$, and $0.7 \times 10^{-3}$, indicating that the increased internal damage in the RC100 group started first.

3.2. Strain Energy Analysis

3.2.1. Dissipated Strain Energy Density and Elastic Strain Energy Density. The essence of concrete damage is the failure of the steady state of concrete driven by various external energies [48]. When concrete is in uniaxial compression in the unconfined pressure state, the total, elastic, and dissipated energy absorbed by the concrete increases with increasing stress before the peak stress [49]. The elastic strain energy is the energy that accumulates within a concrete specimen when it undergoes elastic deformation and is reversible because the elastic deformation is reversible [50]. Dissipated energy mainly consists of: (a) the surface energy consumed during crack opening, development, and penetration; (b) the plastic strain energy consumed by the irreversible plastic deformation of the concrete specimen; and (c) the thermal energy generated by the frictional slip between the cracks and various radiative energies [51, 52]. According to the first law of thermodynamics, it can be obtained that the mathematical

Figure 6: Stress-strain curves of different concrete groups at different ages. (a) The stress-strain curves of RC group. (b) The stress-strain curves of RC20 group. (c) The stress-strain curves of RC60 group. (d) The stress-strain curves of RC100 group.
The relationship between the absorbed total strain energy density $U$ and the elastic strain energy density $U_e$, and the dissipated energy density $U_d$ [53], as shown in the following equations:

$$U = U_d + U_e$$  (1)

$$U = \int_0^\varepsilon \varepsilon \, d\varepsilon$$  (2)

$$U_e = \frac{1}{2} \sigma \cdot \varepsilon^2 = \frac{1}{2E_u} \sigma^2 = \frac{1}{2E_0} \sigma^2.$$  (3)

In the above equation, $\sigma$ is the axial stress, $\varepsilon$ is the axial strain, $E_e$ is the elastic strain, $E_u$ is the modulus of elasticity during unloading, and $E_0$ is the initial modulus of elasticity. Zhou et al. [53] and Li et al. [54] used the initial modulus of elasticity ($E_0$) instead of the unloading modulus of elasticity ($E_u$); therefore, no unloading process is required and the elastic strain energy density $U_e$ is calculated using equation (3). In the test, the modulus of elasticity ($E_0$) was used for the calculation. Figure 7 shows the relationship between the elastic strain energy density $U_e$, the dissipative energy density $U_d$, the initial modulus of elasticity ($E_0$), and the modulus of unloaded elasticity ($E_u$).

### 3.2.2. Variation of the Total Strain Energy Density with the Increase of Strain

In the unconfined compression state (i.e., uniaxial compression), when the stress reaches the peak stress, the damage to the concrete becomes so violent and sudden that the data after the peak stress cannot be accurately recorded [49]. Therefore, only the strain energy density ($U$, $U_e$, and $U_d$) of the rising section of the concrete stress-strain curve in uniaxial compression is investigated. Figure 8 shows the variation curve of the total strain energy density ($U$) with strain for the concrete test set at different ages. It can be seen from Figure 8 that $U$ increases with increasing strain. As the total strain energy density of concrete increases, part of the energy is stored in itself as elastic strain energy and the other part is used as dissipated energy to change its structure. As the age period increases, the ability of the concrete to absorb strain energy increases, which is due to the fact that the longer the curing time, the higher the degree of hydration of the cement, the stronger the internal structure, and the enhanced ability to absorb destructive energy. When the age of curing is 14 days (14 d), the slope of the RC and RC100 curves is greater than the slope of the other groups. This indicates that when the age of curing is short, the degree of hydration of the cement is not high and the brittleness of the RC and RC100 groups is prominent, requiring more external work per unit of deformation than the other groups. The RC20 group has the highest total strain energy density at the peak load when the age of curing is 14 days (14 d), and the slope of the total strain energy curve is lower in the RC20 group than in the RC and RC100 groups, but the strain value at the peak load is higher than in these two groups. This indicates that the RC20 group reduced its brittle properties and accumulated its total strain energy density by increasing its deformation capacity at the age of 14 days (14 d). This also verifies the inference in Section 3.1: “At the age of curing (14 d), the RC20 group mitigates the brittleness by improving its own deformation properties.”

When the curing age is 28 days (28 d) and the strain is less than $0.4 \times 10^{-3}$, the RC curve lies at the bottom. However, as the strain increases, the RC100 curve lies at the bottom and the total energy obtained at the same strain is lower than the other test groups. This is because at 28 days, the cement is more hydrated and the internal cement hydration structure is more solid. By this time, the presence of microcracks and pores within the recycled aggregate is highlighted as thin areas (as seen from the SEM photographs in Section 2.2: the presence of cracks within the recycled aggregate). Compared with the higher stiffness that the cement-hydrated structure has, the weak areas of recycled aggregate have a lower stiffness. The work done externally first closes the primary cracks in these weak areas and causes the pores to be compressed. Above the part of the external work we define as $W_a$. While the test group mixed with rubber powder particles can also absorb external energy (defined as rubber powder deformation energy, $W_p$) through the deformation of the rubber powder after absorbing the external energy $W_a$, which is shown by the fact that when the strain is small, the curve of the RC group not mixed with rubber powder is located at the bottom side of the rubber powder test group. As the particle size of 100 mesh rubber powder is much smaller than 20 mesh and 60 mesh rubber powder, when the quality is the same, the number of particles of 100 mesh rubber powder will be plenty, which will form an extremely large number of weak interfaces inside the recycled concrete. When a certain deformation of RC100 group happens due to the force, the internal damage will increase sharply, and the ability of RC100 group to absorb the damage energy decreases at this time; therefore, it shows that the RC100 curve is located at the bottom with the increase of strain ($\sigma > 0.4 \times 10^{-3}$).

When the curing age is 90 days (90 d), it is found that the difference between the energy values of the four groups of curves is small before the strain was $0.8 \times 10^{-3}$; after the strain is $0.8 \times 10^{-3}$, the energy difference of the test groups
become larger though. For example, at a strain of $0.3 \times 10^{-3}$, the maximum energy difference is $a = 0.000501725 \text{ MJ/m}^3$; at a strain of $0.9 \times 10^{-3}$, the maximum energy difference is $b = 0.00196694397 \text{ MJ/m}^3$; and at a strain of $0.9 \times 10^{-3}$, the percentage of energy differences are 8.5% (RC group), 9.3% (RC20 group), 8.6% (RC60 group), and 8.6% (RC100 group).

It indicates that after a strain of more than $0.8 \times 10^{-3}$ in the test group with a conservation age of 90 d, the damage within the test group begins to increase to varying degrees, as evidenced by a large difference in total energy density beginning to appear.

### 3.2.3. Variation of $U_e$ and $U_d$ with the Increase of Strain

Figure 9 shows the curves of the elastic strain energy density ($U_e$) and dissipative strain energy density ($U_d$) with strain for the concrete test set at different ages. As can be seen from Figure 9, both $U_e$ and $U_d$ show an upward trend as the strain increases. However, at the same strain, the dissipative energy density of the test group is smaller than the elastic energy density. It demonstrates that before the peak stress, the concrete mainly carries out the storage of elastic strain energy. By analysing the $U_e$ and $U_d$ curves, it is found that the $U_e$ curve was similar to the rising section of the stress-strain curve, while the $U_d$ curve was a reverse L curve; that is, when the $U_e$ curve strain developed from 0 to a certain strain value ($\varepsilon_d$), the $U_d$ value continued to be small and almost zero; and when the strain exceeded this strain value ($\varepsilon_d$), the $U_d$ value increases rapidly. We define $\varepsilon_d$ as the value of the dissipated energy sudden strain before the peak load, as detailed in Table 3. The reason for this is that in the early stages of strain development, a small proportion of the external work done is used to close the concrete’s own cracks and compress the pores, resulting in a small amount of

![Figure 8](image-url)  
**Figure 8**: Total strain energy density curves of different concrete groups. (a) $U$ of different groups at 14 d. (b) $U$ of different groups at 28 d. (c) $U$ of different groups at 90 d.
Figure 9: $U_e$ and $U_d$ curves of different concrete groups. (a) $U_e$ of different groups at 14 d. (b) $U_d$ of different groups at 14 d. (c) $U_e$ of different groups at 28 d. (d) $U_d$ of different groups at 28 d. (e) $U_e$ of different groups at 90 d. (f) $U_d$ of different groups at 90 d.
dissipated energy, most of which is used as elastic energy stored in the concrete. As the strain increases to $\varepsilon_d$, damaged cracks within the concrete proliferate and intertwine, leading to an increase in dissipated energy. The crack tip produces acoustic emission energy due to the stress concentration effect, accompanied by irrecoverable plastic strain energy and various sources of radiant energy [50]. At this point, the proportion of dissipated energy is increasing and the proportion of elastic energy is decreasing. The slope of the $U_e$ curve decreases when the strain exceeds $\varepsilon_d$. The smaller the $\varepsilon_d$, the earlier the internal cracking of the concrete, and the more detrimental to the stability of the concrete structure. If the strain exceeds $0.8 \times 10^{-3}$ at an age of curing of 90 d, the $U_e$ and $U_d$ curves change significantly, indicating that the internal damage to the concrete increases and the stability of the concrete deteriorates. It also verifies Section 3.2.2: after the strain exceeds $0.8 \times 10^{-3}$ for the 90-d age of curing test group, the internal damage of the test group starts to increase to different degrees, which is manifested by a large difference in the total energy density starting to appear. Also from Figure 9 and Table 3, it can be seen that at 14 d, the $\varepsilon_d$ of the RC and RC20 groups was higher than the other groups. At 28 d, the $\varepsilon_d$ of the RC100 group was the smallest. At 90 d, the $\varepsilon_d$ of the RC and RC20 groups was higher than the other groups. Combining the $\varepsilon_d$ at the three ages, the best structural stability of the rubber powder recycled concrete was the RC20 group.

### 3.3. Pore Structure Analysis and Fractal Theory Analysis

#### 3.3.1. Pore Structure Analysis
In some studies of the effect of pore structure on the mechanical properties of concrete, the RapidAir457 stomatal structure analyzer is mostly used to determine the proportional distribution of pore chord length (pore diameter), air content, and other factors in concrete [55–57]. The RapidAir457 stomatal structure analyzer was used to determine the percentage distribution of pore chord lengths in concrete specimens, which can be obtained for different chord lengths. The distribution of pore structure in the concrete test set at different ages is shown in Figure 10. In a study by scholar Li et al. [58] on the influence of pore structure on the compressive and splitting tensile properties of concrete, the pore structure measured by RapidAir457 stomatal structure analyzer was divided into three sections: the small pore (<40 $\mu$m), the medium pore (40 $\mu$m–140 $\mu$m), and the large pore (>140 $\mu$m). In this test, the pores are classified as follows: the small pore (<50 $\mu$m), the medium pore (50 $\mu$m–200 $\mu$m), and the large pore (>200 $\mu$m) according to the classification of pores in the above study and combined with the pore test results of the test.

As can be seen from Figure 10, the percentage of small pores in the recycled concrete increases significantly when rubber powder is incorporated, while the percentage of pores in the range of 250 $\mu$m–500 $\mu$m in the large pores decreases. Taking the curing age of 28 d as an example, the percentage of small pores in RC (28), RC20 (28), RC60 (28), and RC100 (28) were 32.5%, 51.9%, 49.1%, and 37.8%, respectively. The percentage of pores in the range of 250 $\mu$m–500 $\mu$m was 12.88%, 6.06%, 8.70%, and 12.50%, respectively. This indicates that the incorporation of rubber powder can refine the concrete pore structure so that the large pore size pores evolve towards the medium or small pores. Firstly, the rubber particles can seal the large pores in the recycled concrete and block the micro cracks formed by internal stresses. At the same time, the nonconstruction material characteristics of rubber powder make it difficult to mix rubber powder concrete, which introduces a small amount of air. After mixing, the air is broken

### Table 3: The actual numerical value of $U_{et}$, $U_{dt}$, $U$, and $\varepsilon_d$.

| Group  | Age (d) | $U_{et}$ (JM/m$^3$) | $U_{dt}$ (JM/m$^3$) | $U$ (JM/m$^3$) | $\varepsilon_d$ (10$^{-3}$) |
|--------|---------|---------------------|---------------------|----------------|-------------------------|
| RC     | 28      | 0.0140444           | 0.0060996           | 0.0201441      | 0.6                     |
| RC20   | 90      | 0.0130857           | 0.0078097           | 0.0208954      | 0.7                     |
| RC60   | 14      | 0.0070887           | 0.0059462           | 0.0130350      | 0.4                     |
| RC100  |         | 0.0095059           | 0.0096494           | 0.0191554      | 0.5                     |
| RC     | 28      | 0.0162739           | 0.0088094           | 0.0250833      | 0.7                     |
| RC20   |         | 0.0184669           | 0.0074564           | 0.0259233      | 0.4                     |
| RC60   |         | 0.0169822           | 0.0088770           | 0.0258592      | 0.8                     |
| RC100  |         | 0.0147657           | 0.0130980           | 0.0278638      | 0.2                     |
| RC     | 28      | 0.0340579           | 0.0078175           | 0.0410755      | 1.0                     |
| RC20   |         | 0.0281207           | 0.0035724           | 0.0316931      | 0.9                     |
| RC60   |         | 0.0267043           | 0.0165403           | 0.0432447      | 0.8                     |
| RC100  |         | 0.0230276           | 0.0152688           | 0.0382964      | 0.7                     |
up and refined, forming a large number of tiny air bubbles, which form tiny pores after the concrete has hardened, which is the reason why the native pore structure of the recycled concrete is further refined.

As the curing age increased from 14 d to 90 d, the percentage of small pores in the recycled concrete increased. For example, in the RC and RC20 groups, the percentage of small pores was 32.1% and 50.4%, respectively, at 14 d. At 28 d, the percentage of small pores was 32.5% and 51.9%, respectively. At 90 d, the percentage of small holes was 35.4% and 58.1%, respectively. This is due to the continuous production and filling of pores by cement hydration products as the curing age increases, causing the pores to be further refined. As a result, the percentage of small pores in the recycled concrete was increased.

3.3.2. Fractal Theory Analysis. Fractal theory is widely used in representing the structural characteristics of concrete pores [59]. The fractal dimension of the pore structure
Figure 11: Continued.
calculated from fractal theory can then be used to some extent to represent the macroscopic mechanical properties of concrete [60, 61]. The most common method for calculating the fractal dimension is the box counting method [58]. It is defined as a series of circular boxes of the same size (box diameter r) to cover the pores, and the number of boxes is N. When the box diameter tends to zero, the ratio of the logarithm of the number of boxes to the logarithm of the inverse of the size is the box dimension. Equations (4)-(5) are formulas for calculating the fractal dimension [62]. It can be seen that lgN and lg (r) present a linear relationship with a slope of D (fractal dimension). Therefore, N at different aperture diameters was converted and a lgN versus lg (r) coordinate system was established to determine D by linear regression, which are shown in Figure 11.

\[
D = \lim_{r \to 0} \frac{\log N}{\log (1/r)}, \quad (4)
\]

\[
\log N = D \log (1/r) + C = -D \log (r) + C. \quad (5)
\]

The fractal dimensions of the three pore size distribution intervals D1, D2, and D3 were calculated according to the classification of pores in Section 3.3.1 and based on (5). The specific calculated values for D1, D2, and D3 are shown in Table 4. The analysis of the data revealed that the pores with a chord length of r = 10 μm (i.e., lg (r) = 1) did not satisfy the overall pore variation characteristics and deviated far from the overall pore, so they were judged to be: singularity and were not considered.

The simplest pore structures have the lowest fractal dimension, while irregular, complex, and inhomogeneous pore structures have a higher fractal dimension [63]. As can be seen from Table 4, the fractal dimension of small and medium pores (D1 and D2) is significantly greater in the RC20 and RC60 groups than in the RC group under the three curing ages. It indicates that the mixing of 20 mesh and 60 mesh rubber powder can enhance the inhomogeneity and complexity of small and medium pores of recycled concrete. It can also be seen from Table 3 that the rubber powder test group has a smaller dimensional number D3 than the RC group. It indicates that the blending of rubber powder reduces the structural complexity of the large pores of the recycled concrete and makes the pore structure simple. Some studies have shown that large pores have a negative impact on the compressive strength of concrete [64]. The admixture of rubber powder reduced the complexity of the macropores and made them more regularly distributed within the concrete. It makes it easier for the damage forces to be transmitted through the large pores when the concrete is damaged by compression, as shown by the reduction in compressive strength.

3.4. Grey Correlation Analysis

3.4.1. Grey Correlation Theory and the Grey Correlation Model. Grey theory was first proposed by Chinese scholar...
Deng [65] in the 1980s. The theory takes an uncertain system with little data and poor information formed by a combination of known partial information and unknown partial information as the object of study. The study is based on macroscopic or microscopic geometric approximations between the behaviour factors in the system [66]. If there is a high degree of simultaneous variation between two factors, the two factors are closely related, that is, the degree of association is high, and vice versa, the degree of association is low [67]. Traditional regression analysis requires that the sample has a large number of reference data and conforms to a standardized distribution equation [68]. As a result, grey correlation analysis theory is widely used in data-poor systems. Grey correlation analysis is a branch of grey theory that considers that the trend of change in a system varies between systems with different values of factors and indicators, and that the degree of similarity in the trend of change between systems is called the degree of correlation. Thus, the degree of correlation is an indicator that describes the degree to which a system is related. This method allows data series to be compared as a whole. The calculation model consists of the following steps:

(a) Determine the reference sequence $Y_i$ and the comparison sequence $X_i$, see (6). The reference series $Y_i$ is a data series reflecting the behaviour of the system, and the comparison series $X_i$ is a data series consisting of factors that influence the behaviour of the system.

$$Y = \{y(k)|k = 1, 2, n\}, X = \{x_i(k)|k = 1, 2, n\},$$

$$i = 1, 2, \ldots, m.$$  \hspace{1cm} (6)

(b) Dimensionless treatment of variables. In this paper, the initial value method is used to dimensionless the data, and $X_i(1)$ is found to be the nonzero data in this test data, so $X_i(1)$ data are used as the reference (see Section 3.4.2: Dimensionless processing of variables for details). The calculation is shown in the following equation:

$$x = \frac{X_i(k)}{X_i(1)} k = 1, 2, n; i = 1, 2, m.$$  \hspace{1cm} (7)

(c) Calculating correlation coefficient. The calculation is shown in the following equation:

$$\lambda_i(k) = \frac{\min, \min_{1 \leq k \leq n} |y(k) - x_i(k)| + \rho \max, \max_{1 \leq k \leq n} |y(k) - x_i(k)|}{|y(k) - x_i(k)| + \rho \max, \max_{1 \leq k \leq n} |y(k) - x_i(k)|}.$$  \hspace{1cm} (8)

$\rho$ is the discrimination factor, generally taken as $\rho = 0.5$ [69].

(d) Calculating degree of correlation. The calculation is shown in the following equation:

$$y_i = \frac{1}{n} \sum_{k=1}^{n} \lambda_i(k), k = 1, 2, n.$$  \hspace{1cm} (9)

3.4.2. Calculation of Grey Correlation Degree. The fractal dimension can reflect the complexity of the pore structure in concrete. In Section 3.3.1, the pore sizes were classified into the small pore (0–50 $\mu$m), the medium pore (50–200 $\mu$m), and the large pore (>200 $\mu$m) according to their size, and the fractal dimensions $D_1$, $D_2$, and $D_3$ were derived for the three pore size ranges. The grey correlation is now calculated to determine which of the three pore sizes has the greatest effect on the elastic strain energy density ($U_{el}$), dissipative energy density ($U_{d}$), total energy density ($U$), and dissipative energy sudden strain value ($\varepsilon_d$) of the concrete at the peak load.

An example of a concrete curing age of 14 days is used for the calculation. The reference sequence $Y_i$ and the comparison sequence $X_i$ are first determined. The elastic strain energy density ($U_{el}$), the dissipative energy density ($U_{d}$), the total energy density ($U$), and the dissipative energy abrupt strain value ($\varepsilon_d$) before the peak load are defined as the reference sequences $Y_1$, $Y_2$, $Y_3$, and $Y_4$. As can be seen from Table 3, the reference sequence $Y_i$ is $Y_i(14d) = (0.0140444, 0.0130857, 0.0070887, 0.0095059)$ for a maintenance age of 14 days. The fractal dimensions $D_1$, $D_2$, and $D_3$ are defined as the comparative sequences $X_1$, $X_2$, $X_3$, and $X_4$, respectively, and the comparative sequence $X_i$ is $X_i(14d) = (0.51845, 1.19966, 1.29380, 1.540446)$ for an age of 14 days. Both $X_2(14d)$ and $X_3(14d)$ can be obtained from Table 4. Placing the raw data is shown in Table 5.

Dimensionless treatment of Table 5 according to (7) yields Table 6. Relating coefficients to Table 6 according to (8) yields Table 7.

The number of correlations was calculated for Table 7 according to (9) to obtain Table 8. The correlations of influences $D_1(14d)$, $D_2(14d)$, and $D_3(14d)$ to $U_{el}(14d)$ are shown in Table 8.

The greater the correlation, the greater the impact of influences $D_1(14d)$, $D_2(14d)$, and $D_3(14d)$ on $U_{el}(14d)$. Table 8 shows that the most influential factor on $U_{el}(14d)$ is $D_3(14d)$. This means that the complexity of the pores with a pore size greater than 200 $\mu$m has the greatest effect on the elastic strain energy density at the peak load at the curing age of 14 days. Based on the above calculations, the correlations of the comparative sequences $X_1$, $X_2$, and $X_3$ to the reference sequence $Y$ are shown in Table 9.
Table 6: The nondimensional initial data.

| Name   | No. | Sequence |
|--------|-----|----------|
| $X_1$ (14 d) | 1   | 2.31393577 |
| $X_2$ (14 d) | 1   | 3.0831506  |
| $X_3$ (14 d) | 1   | 1.042453467|

Table 7: The correlation coefficient.

| Name   | No. | Sequence |
|--------|-----|----------|
| $X_1$ (14 d) | 1   | 2.495515479 |
| $X_2$ (14 d) | 1   | 1.232414477|
| $X_3$ (14 d) | 1   | 0.841177665|

Table 8: The correlation degree of $U_{et}$ (14 d).

| Influence factor | $D_1$ (14 d) | $D_2$ (14 d) | $D_3$ (14 d) |
|------------------|--------------|--------------|--------------|
| Correlation degree | 0.62084      | 0.78362      | 0.84628      |

Table 9: The maximum influencing factor and the corresponding correlation degree.

| Age (d) | $U_{et}$ | $U_{di}$ | $U$ | $\epsilon_d$ |
|---------|----------|----------|-----|---------------|
| 14      | $D_1$ (14 d); 0.84628 | $D_1$ (28 d); 0.78928 | $D_1$ (14 d); 0.81162 | $D_1$ (14 d); 0.85842 |
| 28      | $D_1$ (28 d); 0.79503 | $D_1$ (14 d); 0.80402 | $D_1$ (14 d); 0.78711 | $D_1$ (14 d); 0.79829 |
| 90      | $D_1$ (28 d); 0.76304 | $D_1$ (14 d); 0.86945 | $D_1$ (14 d); 0.68602 | $D_1$ (14 d); 0.7701   |

Figure 12: Imaging analysis by SEM. (a) The SEM of pores in cement mortar. (b) The SEM of damage cracks in recycled aggregate. (c) The SEM of rubber powder and native cracks. (d) The SEM of rubber powder.
sequences $Y_1$, $Y_2$, $Y_3$, and $Y_4$ were calculated and the maximum influencing factors with the corresponding correlations are summarized in Table 9.

As can be seen from Table 9, the highest correlation to $U_{el}$ is $D_3$ at the three curing ages, which indicates that the complexity of the large pore size ($>200 \mu m$) in the test group has a greater influence on the elastic energy density ($U_{el}$) that the test group has at the peak load. It can also be seen that the greatest influence on $\varepsilon_D$ is $D_2$ (complexity of the medium pores) and the greatest influence on $U$ is $D_1$ (complexity of the large pores).

3.5. Imaging Analysis by Scanning Electron Microscopy. Figure 12(a) shows the pores formed within the concrete after it has hardened, which can be seen as a result of air being introduced into the rubber powder concrete due to it not being easily mixed, and the pores being formed after the cement has hardened. It verifies the statement in Section 3.3.1 that "the nonconstruction material characteristics of rubber powder make it difficult to mix rubber powder concrete and thus introduces a small amount of air." Figure 12(c) shows that the presence of the rubber powder acts as a barrier to the formation of primary joints in the concrete due to dry shrinkage. It confirms the statement in Section 3.3.1 that "rubber particles can interrupt microcracks in recycled concrete due to internal stresses." Figure 12(b) shows the damage cracks within the recycled aggregate. Figure 12(d) shows that the rubber powder particles are located inside the cementitious sand, so that when the concrete is stressed, the rubber powder can absorb external energy through deformation.

4. Conclusion

(1) As the age period increases, the ability of the rubber powder recycled coarse aggregate concrete to absorb strain energy increases, which is due to the fact that the longer the curing time, the higher the degree of hydration of the cement, the stronger the internal structure, and the enhanced ability to absorb destructive energy. At 28 days, the cement is more hydrated and the internal cement hydration structure is more solid. By this time, the presence of microcracks and pores within the recycled aggregate is highlighted as thin areas. Compared with the higher stiffness that the cement-hydrated structure has, the weak areas of recycled aggregate have a lower stiffness. The work done externally first closes the primary cracks in these weak areas and causes the pores to be compressed.

(2) Before the peak stress, the rubber powder recycled coarse aggregate concrete mainly carries out the storage of elastic strain energy. In the early stages of strain development, a small proportion of the external work done is used to close the concrete’s own cracks and compress the pores, resulting in a small amount of dissipated energy, most of which is used as elastic energy stored in the concrete. This results in a value of $U_r$ that is essentially similar to the total energy value $U$ in the early stages of strain development. When the strain value reaches $\varepsilon_D$, cracks are produced by internal breakage of the concrete, and as the work done by the external load increases, cracks continue to be produced and intertwine with each other leading to the dissipation of some of the surface energy.

(3) The incorporation of rubber powder can refine the pore structure of the rubber powder recycled coarse aggregate concrete so that the large pore size pores evolve towards the medium (50–200 $\mu m$) or small pores ($<50 \mu m$). The rubber particles can seal the large pores in the recycled concrete and block the microcracks formed by internal stresses. At the same time, the nonconstruction material characteristics of rubber powder make it difficult to mix rubber powder concrete, which introduces a small amount of air. After mixing, the air is broken up and refined, forming a large number of tiny air bubbles, which form tiny pores after the concrete has hardened, which is the reason why the native pore structure of the recycled concrete is further refined.

(4) The mixing of 20 mesh and 60 mesh rubber powder can enhance the inhomogeneity and complexity of small and medium pores of recycled concrete. The admixture of rubber powder reduced the complexity of the macropores and made them more regularly distributed within the concrete. It makes it easier for the damage forces to be transmitted through the large pores when the concrete is damaged by compression, as shown by the reduction in compressive strength.

(5) The complexity of the large pore size ($>200 \mu m$) in the test group has a greater influence on the elastic energy density. The greatest influence on $\varepsilon_D$ is the complexity of the medium pores (50–200 $\mu m$). The best structural stability of the rubber powder recycled concrete was the RC20 group.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

The authors gratefully acknowledge the financial support of this work provided by the National Natural Science Foundation of China (Grant no. 52069024) and the Major Science and Technology Projects of Inner Mongolia Autonomous Region of China (Grant no. 2021ZD0007).
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