USE OF RECYCLED AGGREGATE IN SHOTCRETE AND CONCRETE

by

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ABSTRACT

As the problem with waste concrete escalates in most industrialized nations, growing efforts have been directed towards recycling this material. Recycling of wastes, besides providing an alternate route for their management, also contributes to the conservation of natural resources. With waste concrete, this is primarily done by crushing the material to desired particles sizes and reusing it as aggregates suitable enough for specific applications. However, to cope with the increasing levels of demolished concrete being generated, more potential applications for its recycling must be sought.

The purpose of this research was to investigate the use of recycled aggregates in shotcrete. Both, wet and dry processes were investigated and compared to companion cast concrete. The mechanisms by which recycled aggregates affect the performance of concrete were also analyzed. For a fair comparison between recycled and virgin aggregates, they were both made to have particle size distributions as close as possible to each other by matching the virgin aggregate gradation to the recycled aggregate gradation using a regression style analysis. The resulting mixes were investigated for their fresh properties as well as hardened properties.

In the wet process, recycled aggregates were found to produce a quick stiffening and a rapid loss of workability of the fresh mixture. This is due to the higher water absorption and cohesiveness of the material due to the higher content of the fine material liberated during the dry-mixing of recycled aggregates. Such properties, however, brought a significant positive contribution to shotcrete in that the rebound was found to be much lower compared to virgin aggregate mixes in both wet-mix and dry-mix shotcrete. Reduction was seen in both material as well as fiber rebound in fiber reinforced shotcrete mixes. While some attention must be paid when shotcreting, because of a different shooting and pumping behaviour of recycled aggregates, casting of concrete with recycled aggregates can be carried out in the normal ways.

Compressive stress-strain tests performed on drilled cores revealed that the fracture process of cylinders with recycled aggregate mixes is very stable and gradual, unlike the unstable and rapid crack propagation in cylinders with virgin aggregates. Although the compressive strengths in recycled aggregate mixes were 40-56% lower than virgin aggregate mixes, the former had greater deformability. This greater deformability, accompanied by stable failure, allows for a much higher energy absorption capacity. Splitting tensile strengths, flexural strengths, flexural toughness of fiber-reinforced mixes and moduli of elasticity of recycled aggregate systems were also found to be lower than virgin aggregate mixes. Microscopic observations revealed that large quantities of dust and loose debris exist in recycled aggregate mixes which are likely a contributing cause to the above observations. It appears that recycled aggregates affect shotcrete mixes in the same way as they affect cast concrete mixes such that there is no apparent gain or loss in using them in shotcrete. Fiber reinforcement of recycled aggregate shotcrete mixes is not as effective as it is with virgin aggregate mixes although suitable enough for many applications. Finally, more research on durability related issues of recycled aggregate systems must be carried out to obtain a more complete understanding of these materials.

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to my parents

1.0 INTRODUCTION

As we move into the 21st century, a major challenge faced by most industries and societies is conservation of the environment. One significant factor contributing to this awareness is raised by the recognition that many of the natural resources on which the humankind depends are being exploited to depletion levels. At the same time, waste management has become a more serious issue considering that its mismanagement can lead to ground and water pollution, disease vectors, spontaneous combustion, and explosion hazards, and other negative environmental impacts. Efforts have therefore been made to find proper ways to manage these wastes while reducing their impact on the environment. Of the three most common methods of waste management - landfilling, incineration, recycling - the third one is emerging as a popular alternative for solid waste handling. Wastes can be thought of as resources that are in the wrong place at the wrong time. With this idea in mind, recycling is receiving greater attention as it could potentially solve both the waste disposal and resource depletion problems. In general, recycling would entail the following basic activities: collecting secondary materials, preparing the materials for market and selling them to manufacturers, and actually using those materials to manufacture new products.

Although waste material recycling can be contemplated as a viable solution to the environmental problems mentioned previously, there are a number of barriers that must be overcome before such processes can be implemented, even if it were technically feasible. These include social constraints such as public acceptance; political considerations; environmental constraints; legislative and regulatory mandates; financial limitations; and organizational structure of the bodies participating in the industry. The main problem has been one of hesitation in putting this technology in practice. Until environmental impacts are not manifested as seriously threatening, governments will not feel pressured to set stricter recycling regulations, industries will continue to take advantage of virgin material available for the quality of their products, and consumers will prefer virgin, new products due to psychological reasons. At some point in time, though, this luxury will cease to exist as many of the environmental problems incurred would be irreversible and too late to take care of. It is therefore of paramount importance that measures be taken while the problem is still manageable, particularly now that societies in general are already aware of these hazards. However, such an endeavour will require efforts from different members in society including governments, industries and the public in general.

In the context of resource and waste material management, concrete, being the most widely used construction material, represents a very significant concern. With the growth in population and urbanization taking place in most countries around the world, concrete has seen its demand rise over the last few decades. At the same time, the restructuring of infrastructure works along with unprecedented disasters have, on the other hand, contributed to also increasing quantities of the waste concrete stream. Such activities, if continued without control over the long term, would place detrimental impacts on the environment. In fact, many countries like Japan, Belgium and the Netherlands have felt this impact as their concrete-making resources are becoming scarce while another valuable resource - land - is also becoming scarce, and too valuable to be given up for use as landfills. As Sakai mentions in his article [1], the 21st century is the century of the environment, and as environmentally conscious engineers, a number of facets in the concrete industry have to be emphasized for stricter control and further investigation and action. One of the main ones is the recycling of waste concrete.

The current state of concrete recycling can be described as being very spasmodic. Only countries with such need and urgency have made serious advancements into it, whereas other countries either have not done anything about it or engage in very limited recycling activities. The general problem holds social, economic and technological constraints. In the low activity countries, it is a combination of mainly, the social and economic constraints that have hindered continued advancement in the technological area. On the other hand, there is a necessity from the technological side to continue searching for more efficient demolition techniques, better methods for transporting the wastes and more areas of application, beyond the typical and limited present day uses of it. Through compromise and simultaneous effort into the three constraints, the concept of waste concrete recycling can, hopefully, progress further into a stage where it is executed at high rates and adopted as a norm by most societies. Then, the general environment can be saved from one of its illnesses. The research presented in this thesis looks into an application of recycled aggregates that has seen little or no use yet at all, which is shotcrete. Only by investigating the behaviour of this waste material as a reused material can it be possible to determine the feasibility of such use. Hopefully, in the near future, other applications for recycled aggregates will be devised.

2.0 OBJECTIVES AND SCOPE

As mentioned in the Introduction, waste concrete recycling is a technology that must be developed in the near future in order to preserve the environment. Recycling of any waste is subject to three general constraints: social, economic and technological. Social and economic yardsticks are critical in determining whether or not a concept can be put into practice. These are, however, not encompassed in this research program; the primary purpose here is to investigate the technological aspects of this problem. A very important factor contributing to a targeted increase in recycling rates arguably is the effort towards finding more potential applications for recycled aggregates. The intention of this research is to determine the technical feasibility of an application that has seen little or no use of crushed waste concrete.

This research investigates the use of recycled aggregates in shotcrete. Shotcrete is an area of application that is increasing and, at the same time, consumes large quantities of material. Shotcrete has uses in the repair of deteriorated structures, in the upgrading of outdated structures to modern standards, in rock support, in the construction of concrete shells, and, very importantly, in the mining industry as ground and tunnel support. If part of the materials required for its production can be replaced by a lower priced substitution, it could lead to significant cost savings in this industry, thereby further promoting its use. This is particularly true in regions where the production of recycled aggregates has been well systematized and organized. Apart from cost savings, the use of recycled aggregates in shotcrete may show trends which differ from those observed with normal cast concrete and may even prove to be more beneficial to shotcrete. The premise behind this idea is simple: shotcrete is a pneumatically projected and compacted concrete possessing very different compaction dynamics from conventional cast concrete [12]. As a result of this, the mode of interaction between the mix water, the cementitious part and the filler part of the mix are different; and, a different rheology and spatial distribution of these components within the mix suggests that the kinetics of strength gain and matrix-aggregate interface development in shotcrete are decidedly different from those occurring in cast concrete. It is therefore entirely conceivable that the use of recycled aggregates in shotcrete may produce and even display patterns which are different from those observed in cast concrete. It should be kept in mind that the possible benefits imparted by the incorporation of recycled aggregates in shotcrete do not have to be restricted to the hardened mix properties; there could be benefits realized during the production process itself.

Besides investigating the potential use of recycled aggregates in shotcrete, this research also looks into possible mechanisms by which such aggregates alter the properties of cementitious systems. It is generally known that recycled aggregates lower the performance of concrete, and, it would be interesting to find out the causes for such reductions and, to devise any performance enhancing alterations. For two aggregate sources to be compared for the effects of their inherent nature, they must be investigated free from other possible sources of influence, such as particle size gradation, water absorption, and particle size and shape. Previous research conducted on recycled aggregate concrete has not explicitly considered these variables. This research here also aims at maintaining similarity between aggregate types, recycled and virgin, in order to look at the differences that are actually caused by their distinct nature. Thus, part of the experimental work involves setting similar conditions for both aggregate types. Once done, the effects of aggregate type on the fresh mix properties are examined, followed by an investigation of the effects on various fundamental hardened concrete and shotcrete properties.

3.0 LITERATURE SURVEY

3.1 Stating the Problem and the Need for Recycling

3.1.1 The General View

With an increased growth in population and urbanization in many countries, there has been a similar trend in the demand for the construction of infrastructure. Concrete, being the most widely used construction material, has seen its consumption rise steadily over the last few decades. This can be in houses, from the basement to the tiles on the roof; in the city, from roads to bridges to sewer lines; and in commercial structures, from office complexes to factories. The actual consumption varies from region to region; but, for example, it can have a rate as high as 10 kg per capita per day in a country like Canada [2]. Designers of buildings and other structures normally focus on the erection of such works with little or no attention to the source of the concrete or to the demolition of the works after their technical, economic, or social service life has expired. This ignoring tendency is where many of the problems relating to the manufacture of concrete and its disposal crop up.

The life cycle of concrete can be envisioned as a linear process, where at one end it is produced, at the other end it is disposed, and through the length of the line, it is in its service life. Designers are generally concerned with the performance of concrete and the structure during its service life. Furthermore, this period of time is normally long enough for the industry to overlook the combined problems that arise at the two extremes of the line. However, most of the environmental threats stem from the two ends. As will be discussed in the next sections, the problems that exist at the two ends touch on the current major global environmental concerns like air pollution leading to global warming and acid rain, natural resource depletion, waste generation, contamination, etc. Thus, to minimize these problems, the most obvious way is to try joining the two ends of the line to a single point, thereby looping the process. This leads to the concept of waste concrete recycling which, as Sakai pointed out, is one of the issues concrete technology should emphasize as we enter the century of the environment [1]. Obviously, the execution of a recycling program is subject to financial, economical and political constraints; but, these may have to be compromised if efforts to preserve the environment are to be realized.

3.1.2 Problems on the Concrete Production Side

The production of concrete has a number of significant impacts on the environment that can ultimately affect decisions regarding its recycling. The main ones are described as follows:

1. Energy Consumption

   The production of concrete requires a large amount of energy. The embodied energy for its production is about 2.3 million kJ/m^3 of concrete, most of it from the manufacture of cement (about 3.79 GJ of thermal energy and 0.56 GJ of electrical energy per ton of cement) [3, 4].

2. Consumption of Natural Resources

   The continuous quarrying of limestone, shale, sand and gravel from natural deposits will eventually lead to their depletion, particularly in places where these materials are scarce.

3. Transportation Costs

   The costs of transporting the resources to manufacturing plants represent a significant proportion of the total cost in the production of concrete.

4. Environmental Pollution

   The largest threat from an environmental standpoint comes from cement production. The major impacts of the cement industry on the environment are:

   - emissions of particulate matter and cement kiln dust (CKD), most of which has sizes in the range of a few microns;
   - emissions of carbon dioxide (CO_2), sulphur oxides (SO_x) and nitrogen oxides (NO_x);

- opacity from the plumes expelled;
- energy requirements;

5. Water Pollution

   In the mixing of concrete, the washout water contains an alkalinity with a pH as high as 12. This is toxic to fish and other aquatic life.

3.1.3 Problems on the Concrete Disposal Side

The largest and, perhaps, most important urge for the construction industry to engage concrete recycling comes from the escalating quantities of waste concrete that are generated and in need to be managed. Once concrete has served its purpose as a construction material, it becomes a waste like many other materials which face similar fates. There are various reasons for the eventual wasting of concrete at some time during or after its service life. The most common one is the demolition of concrete structures, such as buildings, sidewalks, pipes, bridges, etc. As societies grow, many of these have to be razed after a period of time into their service life for purposes of replacement or landscape changes. An example of this is shown in Figure 3.1. Other sources of wastes include: natural disasters like earthquakes, tornadoes or floods; human-made disasters like wars and bombings; and structural failures. All these contribute to vast quantities of waste concrete that must be handled in some way. The actual amount of it produced varies from place to place; but, to get an idea of the magnitude of the problem, the following numbers were compiled:

- In the European Economic Communities, approximately 50 million tons of concrete are currently being demolished each year [5];
- In Japan, about 25 million tons of waste concrete were produced in 1990. It was estimated that this amount will rise to 71 million tons by 1995 and to 110 million tons by the year 2001 [6];
- In the United States, this amount is roughly 60 million tons per year [7, 8];

- In the United Kingdom, about 11 million tons of demolished concrete are dumped at landfill sites each year [9].

It is estimated that, by the year 2000, three times more demolished concrete will be generated each year than that generated in 1992 [10]. Given these quantities, it can be foreseen that the construction and demolition industries will come under considerable pressure to find a proper way of handling these wastes.

Figure 3.1 Concrete building in its demolition stage.

Traditionally, the management of waste concrete has been mainly landfilling and partly, recycling. Only a fraction of the total amount of waste stream produced is being recycled, and it is mainly used in low grade applications such as road subbases and fills. The proportion of the total waste stream that is recycled also varies from place to place but generally lies in the range of 10-35% for active regions and 0% for non-active regions [10]. A further complication that arises is the fact that the amount of participation from industry and from the general public is very spasmodic as there is no definitive legislation on this issue. The result of this is that about 65-100% of all the concrete wastes produced go to landfills. This means that, in a place like Japan, this amount could reach over 70-110 million tons in the year 2001; and, in the United States, it could reach 30-50 million tons per year. With the large amount of waste comprising this stream and an assured increase of it in the future, landfilling has become a major problem, particularly in countries where land is scarce. Besides the scarcity of land, other problems facing the landfill option include their siting, transport costs, tipping fees, and public opposition. Thus, recycling has been gaining wider attention as a viable option for the handling of waste concrete.

From the above considerations, including those from the concrete production side, it can be seen that the most sensible way to manage waste concrete is to recycle it into applications where aggregates would be needed. With the increasing quantities produced, these applications must extend beyond their mere use as road subbase or fill material. At the same time, applications into other areas requires that these be proven to be technically sound. Governing bodies must act to provide some pressures and, perhaps, also incentives to the industry in order to find further ways to recycle concrete wastes. An example is to stipulate target recycling rates which must be met within a certain period of time. Certainly, the combined effort of all participants in this industry (government, demolition contractors, consultants and researchers) is required if an effort is to be made to improve the condition of the environment in which we all live and interact.

3.2 Current Practice

Presently, the major markets for recycled aggregates are in [10]:

1. General bulk fill

2. Base fill in drainage projects
3. Sub-base or surface material in road construction
4. New concrete manufacture

For each of the above uses, there are general characteristics required of crushed concrete before these are utilized. Some examples of these characteristics include their shape, hardness, and cleanliness. An application where recycled aggregate has, in particular, seen its greatest use is in the construction of pavements [11]. This is because crushed concrete has a number of properties which are suitable for such usage and the process is easy and economical. For instance, the pavement concrete to be crushed is usually in the exact place where the new pavement will be constructed. This saves on quarrying and transporting costs. Besides, concrete from pavements tends to be more or less consistent in quality and free from contaminants, especially from other building materials. Moreover, failure of a concrete pavement does not have the same potential consequences, including loss of life, as does the failure of concrete in a structure. Crushed concrete pavements have been used as sub-base or base course material, in bituminous mixes, as filter material and as low quality stabilization/filler material in the construction of new pavements. Other minor applications for recycled aggregates include fill for breakwaters and levees, rip rap, ballast, roofing granules, neutralizing beds, filtration beds, thermal reservoirs, sound barriers, masonry and cat litter boxes [11].

There are several incentives for the industry to use recycled aggregates in any of the above applications. These include [12]:

- <u>Cost</u>

  The cost of recycled aggregates compares quite favourably with virgin aggregates in most countries. In Vancouver, Canada, recycled aggregates cost about $8 per ton as opposed to $15 per ton for virgin aggregates, in 1998 [13].

- Municipal Approval

  Most municipalities allow the use of recycled aggregates if they meet certain criteria.

- Waste Management

  With the general need to reduce and manage wastes, the use of recycled aggregates in concrete is one of the most effective ways.

- Lower Densities

  Lower densities of recycled aggregates generally mean lower concrete densities and higher yields.

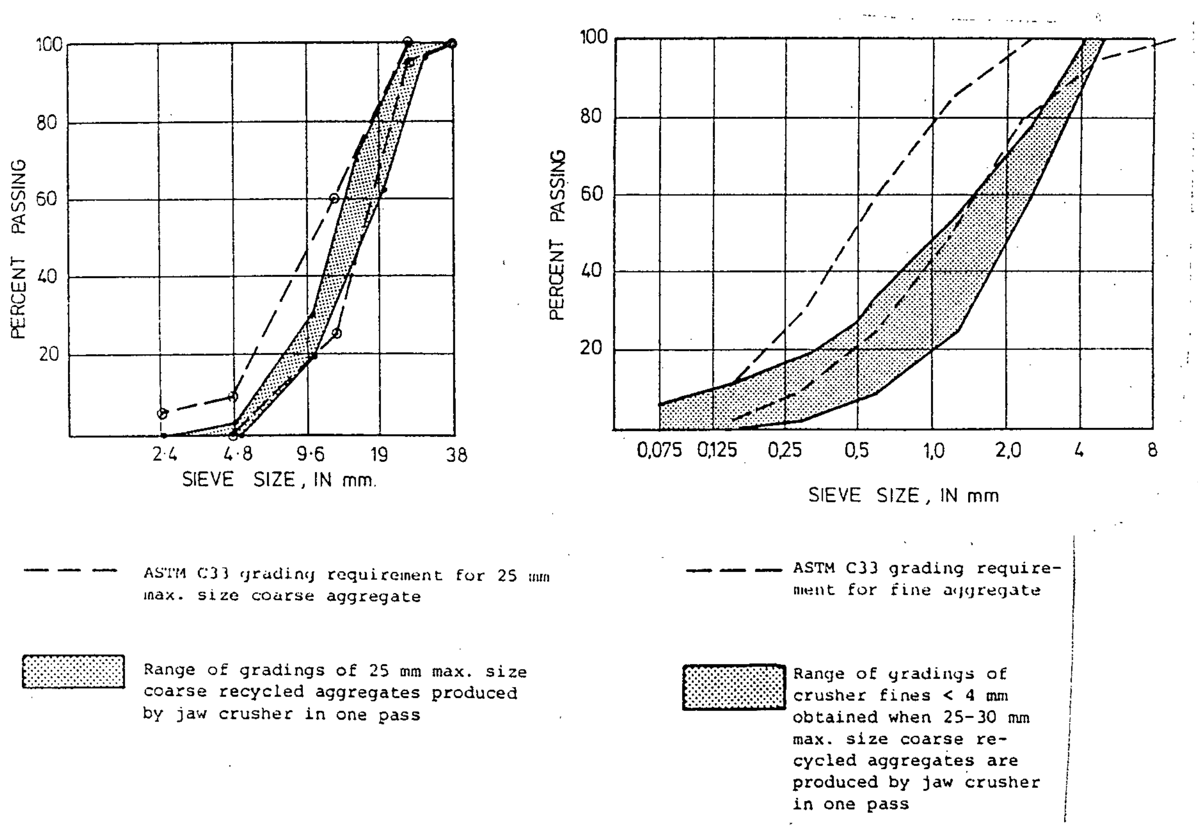
Different countries have different standards and regulations regarding the use of recycled aggregates. Many of these cover the required quality of the crushed concrete, such as the level of contamination, particle size and shape, and absorption. These stipulations are controlled by local conditions, economics and political factors which dictate the strictness in the use and potential applicability of such aggregates. References to these specific standards can be found elsewhere [3, 10, 14, 15, 16, 17, 18].

3.3 Properties of Recycled Aggregates

3.3.1 Particle Characteristics

The physical properties of recycled aggregates are determined, mainly, by the crushing operations and, partly, by the strength of the original concrete. Different countries have different guidelines regarding the properties that recycled aggregates should possess in order for them to be suitable for using in concrete [3, 14, 15, 16, 17, 18]. Most of these properties can be generalized as being inherent to the aggregates due to way in which they are produced.

Since the waste material is passed through crushers (jaw, cone or impact), it is not surprising to expect angular particles with a rough surface texture. The particle shape tends to be more irregular and coarser than that of virgin aggregates [3], which may lead to concrete mixtures that are harsher and less workable. However, in general, the main purpose of the crushing operations is to reduce the size of the aggregates and bring them to within the limits stipulated by ASTM C33 (Standard Specifications for Concrete Aggregates) [19] or other similar standards. This objective has been observed to be best achieved with the jaw crusher as this produces a reasonably well-graded material and can be adjusted to yield a certain maximum size of the particles [10]. Figure 3.2 shows the range of gradations achieved from several experiments in comparison to the ASTM C33 limits for coarse aggregates with a maximum size of 25 mm and for fine aggregates, as compiled by Hansen [10].



(a) (b)

Figure 3.2 Range of gradings produced by jaw crushers: (a) for 25 mm coarse recycled aggregates; and (b) for fine recycled aggregates [10].

Notice that the gradation band for recycled aggregates tends to be fairly uniform and in accordance with the stipulated standards. The fines content of the recycled aggregates, in particular, given its large quantity and angular nature, can create problems in terms of producing a harsh, unworkable mix. This problem can usually be solved by blending some natural sand into the mix.

One of the most significant and generally recognized features of recycled aggregates with large influences on their properties is the presence of attached mortar and cement paste on the surface of the original aggregates. In a cut section of newly cast concrete, this attached mortar can be seen as a lighter, more porous phase extending from the coarse aggregate surfaces. Hansen and Narud [20] found the volume percentage of mortar adhered to natural gravel particles for several size ranges. These are: 25-35% for 16-32 mm coarse recycled aggregates; around 40% for 8-16 mm coarse recycled aggregates; and, around 60% for 4-8 mm coarse recycled aggregates. They also concluded that, for the same cement and original aggregate, the volume percentage of old mortar adhered does not vary much even for widely different water-cement ratios of the original concrete. However, one apparent trend is that the amount of attached mortar increases with decreasing aggregate particle size; thus, finer aggregates contain more of this attached mortar. That is, it increases with available surface area. Nojiri [21] also determined that the amount of this surface mortar depends on the level of crushing. It could range, for a normal strength concrete, from about 30% for just primary crushing down to 3% for a high level of crushing. As will be seen in later sections, this old mortar is the root of the problems when recycled aggregates are used.

3.3.2 Density

Particle density is an important value for mix design and for the accuracy of batching. Variations in the composition and origin of recycled material lead to variations in density values. Hansen and Narud [20] found densities of coarse recycled aggregates in SSD conditions ranged from 2340 kg/m^3 (for 4-8 mm material) to 2490 kg/m^3 (for 16-32 mm material). The corresponding SSD densities of original coarse aggregates ranged from 2500 to 2610 kg/m^3. They also measured the density of old mortar and found them to be around 2000 kg/m^3. This lower value, along with the presence of lighter foreign substances, gives recycled aggregates an overall lower density compared to the virgin aggregates. However, they also reported that these density values would not vary much even for widely different water-cement ratios of the original concrete. Other density values that have been determined range from 2100-2500 kg/m^3 [18, 22, 23, 24, 25, 26]. The bulk density has been reported to be in the range of 1300-1400 kg/m^3 [24].

3.3.3 Water Absorption

Water absorption of an aggregate sample is another important property. Between recycled and virgin aggregates, one of the most marked differences in their physical properties is their different water absorbing capacities. ASTM provides standards for determining this property for coarse and fine aggregates separately [27, 28]. However, as Hansen [10] reports, for recycled aggregates, it is more difficult to determine water absorption capacity and water content of the fine fraction compared to the coarse part. This is because the fine part tends to be fairly sticky making the test inappropriate and highly inaccurate. Proper judgement is needed in assessing this property [10].

Water absorption values for recycled aggregates are always higher than those for virgin aggregates. This is primarily due to the presence of attached mortar and other foreign substances which can absorb more water than normal aggregates. Hansen and Narud [20] found water absorptions of coarse recycled aggregates ranging from 8.7% for 4-8 mm material to 3.7% for 16-32 mm material, regardless of the quality of the original concrete. Corresponding water absorptions of original aggregates ranged from 0.8 to 3.7%; and, for mortars, it was found to be around 17%. Other values that have been reported range from 3-8% for coarse recycled aggregates and from 8-12% for fine recycled aggregates [18, 22, 23, 24, 25]. Hansen suggests that, as a result of the much higher water absorption of recycled aggregates, it may sometimes be more convenient to pre-soak them during the production of recycled aggregate concrete in order to maintain a uniform quality between batches.

3.3.4 Los Angeles Abrasion Loss and British Standard Crushing Value

The Los Angeles abrasion test and British crushing test are used to determine the wear resistance of aggregates, as cement paste has little resistance to abrasive conditions. This is important from the standpoint of strength and durability [29]. Hansen and Narud [20] found Los Angeles (LA) abrasion loss percentages ranging from 22.4% for 16-32 mm coarse recycled aggregate produced from a high strength original concrete, to 41.4% for 4-8 mm coarse recycled aggregate produced from a low strength original concrete. Corresponding LA uniformity numbers L100/L500 were 0.24 and 0.38. BS aggregate crushing values were 20.4% and 28.2%, respectively. Other abrasion loss numbers reported range from 25-42%, this value dropping with a corresponding higher strength of the original concrete [20, 23, 24, 25]. According to the ASTM C33, aggregates may be used for production of concrete when the LA abrasion loss percentage does not exceed 50%. Crushed stone for road construction purposes is usually required to have LA loss values not exceeding 40%. Thus, recycled aggregates generally satisfy both these requirements.

3.3.5 Presence of Contaminants

One of the main problems in the recycling of waste concrete is the existence of foreign substances, known as contaminants. These generally occur from the original demolition debris which gets passed on to the waste concrete. The problem with these contaminants is that they possess an array of different physical and chemical properties, many of which are incompatible and deleterious to the performance of the resulting new concrete. The following is a list of some of the major contaminants which may exist in appreciable quantities in waste concrete [10, 30].

- metals such as reinforcing steel or non-ferrous metals such as aluminum, lead, copper, brass and bronze;
- brick which may contain a high content of periclase, MgO;
- organic material like wood, textiles, and paper;
- gypsum from drywall;
- glass;
- bitumen from asphalt;
- plastics;
- soil and filler materials;
- chemical and mineral admixtures which may still be reactive from the original concrete;
- chemical contamination, for example, from concrete used in nuclear reactors or wastewater treatment plants, or chlorides from de-icing salts.

Because of the varying nature of the contaminants, the properties of the new concrete can be potentially affected either in the fresh state or in the hardened state. The higher absorptions of some of these contaminants like soil and brick could also lead to workability problems. Non-uniform particle shapes and textures can also have an effect on workability. In the hardened state, the softer and less bonding nature of the majority of these foreign substances leads to significant reductions in the mechanical properties of the new concrete. Further, because of their reactive nature, unsoundness and porosity, long term durability problems are likely to prevail in the new concrete. Gypsum, for example, is a main concern as its solubility and reactivity with aluminous compounds can lead to very fast deterioration of the concrete material. Further details on the potential damage the contaminants could have on the new concrete can be found elsewhere [10, 30].

Different separations techniques are employed to remove the contaminants. However, because of the various different types of contaminants present, there is no single method that can get rid of them all. Some minor quantities will always remain in the processed aggregates.

3.4 Mix Design of Recycled Aggregate Concrete

In the design of concrete mixes using recycled aggregates, either as a total or as a partial replacement of the aggregate fraction, the most significant consideration is the higher water absorption these aggregates have compared to virgin ones. With absorption rates as high as 8% for coarse recycled aggregates and 12% for fine recycled aggregates (Section 3.3.3), it is evident that extra water must be added to compensate for the amount of water taken up by them. The extra amount needed depends on the aggregate replacement level. Concrete mixes with a higher fraction of fine recycled aggregates would thus require additional water. It has been reported [20, 31] that recycled concretes produced with coarse recycled aggregates and natural sand require approximately 5% more free water, in order to achieve the same slump as control mixes. The corresponding number when both, coarse and fine, recycled aggregates are used as replacements can be as high as 15%.

To counteract the additional water demand of recycled aggregates during the production of concrete, there are three different methods that can be employed in the field. These are [18]:

1. increasing the supplementary water addition according to the 30 minute absorption test;
2. saturating the aggregates by pre-soaking (normally, 10-15 minutes depending on aggregate size);
3. adding water, superplasticizers or water reducers at the construction site according to the required workability.

The second method would be more accurate in theory; but, it is not desirable in practice due to the additional arrangements needed on site. While the first method is of sufficient accuracy, the third one is preferred at site because workability has a first priority. Regardless of the method employed, there is little or no difference in the resulting strengths of the hardened concrete, as long as the free water-cement ratios of the fresh concretes are the same. The concretes containing recycled aggregates would, however, experience rapid slump loss and an earlier set, especially if produced with dry aggregates. This is caused by the much higher absorption of the fine part [20, 32].

Another process that can be carried out during the production of recycled aggregate concrete is dry-mixing of the aggregates. In this process, the aggregates are placed in a mixer and allowed to mix for a certain period of time prior to the addition of the other concrete making ingredients. This way, the attached mortar can be ground off, while a finer grading of aggregates is produced. The workability of concretes in the fresh state that have been dry-mixed may be worse because of the presence of additional fine material but may also be better due to a better particle shape attained in the process [32, 33]. To compensate for the higher absorption, the aggregates can be pre-saturated, just as long as the free water-cement ratio is maintained constant.

As far as the fine recycled aggregates are concerned, Hansen [10] suggests that they not be used for the production of new concrete at all because of the high water absorption and cohesiveness, and the difficulty in determining the absorption rate making it harder to control the quality of the concrete being produced. Moreover, the fine fraction of recycled aggregates has detrimental effects on many of the hardened concrete properties (Section 3.5). Therefore, some screening process would be required to separate the fines, which is defined as that fraction finer than 4-5 mm, from the coarser part.

Besides the above considerations, the mix design process of recycled aggregate concrete is no different from the mix design of conventional concrete, and the same methods can be used. In practice, slight modifications are required. The DOE method [34], which is widely used in the United Kingdom, stipulates the following modifications:

1. Setting a higher standard deviation when determining a target mean strength;
2. Assuming that the free water-cement ratio for a required compressive strength will be the same for recycled aggregate concrete as for conventional concrete when coarse recycled aggregates are used with natural sand, unless trial mixes prove otherwise;
3. Assuming that, for the same slump, the free water requirement of recycled aggregate concrete is 10 l/m^3 higher than for conventional concrete;
4. Using a maximum recycled aggregate size of 16-20 mm for durability reasons;
5. Adjusting the cement content slightly upwards to accommodate the additional water required;
6. Designing the mix based on the measured density of the recycled aggregate at hand;
7. When estimating the ratio of fine to coarse aggregate, assuming that the optimum grading of recycled aggregates is the same as for conventional aggregates;
8. Making trial mixes to adjust the free water content to obtain the necessary slump.

Recycled aggregate concrete also displays other fresh concrete properties different than those of conventional aggregate concrete. These include a lower density, a higher air content, and lower bleeding [10].

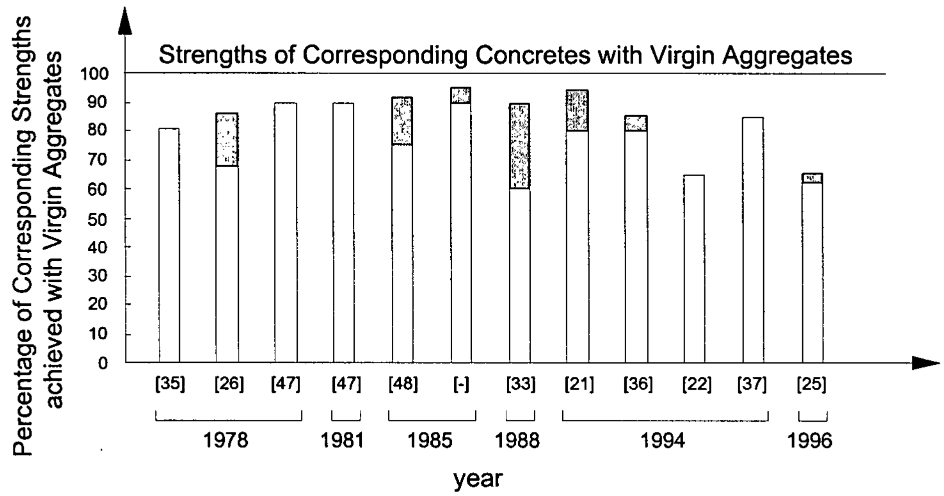
3.5 Mechanical Properties of Recycled Aggregate Concrete

3.5.1 Compressive Strength

The compressive strength of concrete is the most widely used single indicator of its quality. For this reason, almost all the tests carried out on recycled aggregate concretes measure their compressive strengths and compare them to corresponding concretes made with virgin aggregates. As mentioned in Section 3.4, the fine fraction (less than 4-5 mm) of the recycled aggregates causes a number of detrimental effects on the properties of the new concrete, in both the fresh and in the hardened state; Hansen [10] even suggests excluding the fine portion from the mix. Thus, most of the research done on recycled aggregates has focused on replacing the coarse fraction completely and replacing the fine fraction at either, zero or several percentage levels. These two types of fine aggregate substitutions will be discussed here.

Although the compressive strength value of recycled aggregate concrete is important, a parameter that is equally important and one that has been reported more often, is the percentage decrease in the strength when recycled aggregates are used to replace virgin aggregates [10]. This is because, with natural aggregates, good control can be exercised over the performance required. Therefore, the percentage reduction in strength values gives an indication of the reduction in performance that a certain batch of recycled aggregates can inflict on the new concrete relative to virgin aggregates. From a collection of works, it has been gathered that, when the coarse aggregate fraction is completely replaced by coarse recycled aggregates, the compressive strengths of the resulting concretes have ranged from as low as 15 MPa up to 60 MPa, for a wide range of water-cement ratios of the mix and different strengths of the original concretes [10, 20, 21, 22, 25, 26, 33, 35, 36, 37]. At the same time, in the concretes investigated, the percentage reduction in strength compared to the corresponding concretes made with virgin aggregates ranges from 5-40%, with one of them reported as being as high as 80%. An interesting observation is to trace the changes in these values over the years. This enables one to see if any improvements in technology have been made based on a better understanding of the underlying problems of using recycled aggregates. From the data gathered, Figure 3.3 is plotted.

It is apparent that over the years, not much improvement has been made in developing better strengths from the recycled aggregate concretes. Large variations in strength still exist throughout the 20 year period covered. One the possible reason for this, as proposed by Hansen and Narud [20], is that the attainable strength of the new concrete mix depends, by and large, on the strength of the old concrete. They demonstrated this



Note: shaded part of bars represents a range of results
[#] is the reference number

Figure 3.3 Strengths of recycled aggregate concrete as a percentage of the strength of corresponding virgin aggregate concretes over time (recycled coarse aggregates and virgin fine aggregates).

by making concretes of different strength levels, crushing them, and using them as aggregates to make concretes with the same strength levels as the original concretes. If the strength of the original concrete is the same or higher than that of the recycled aggregate concrete, then the strength of the recycled aggregate concrete can be as high or even higher than the strength of the original concrete. This, in turn, leads to a lower percentage decrease or even to a percentage increase in strength when using recycled aggregates. In practice, perhaps, the variability and lack of control over the strength of the original concretes, in addition to the variable states in which recycled aggregates are found and the presence of the many different contaminants are all reasons why there is no definite means by which a given batch of recycled aggregates can be used to produce a concrete of specified strength. Parameters such as the water-cement ratio and cement content can be adjusted to obtain better strengths; but, the variability of the material will always present an uncertainty factor with respect to strength. This variability therefore can be considered a limitation when trying to optimize systems containing recycled aggregate concrete.

The results for systems where both, the coarse and the fine, aggregate fractions are replaced by recycled aggregates have also been compiled from a collection of works [10, 21, 24, 25, 26, 33, 36, 37, 38]. Also, from these, the strengths as well as the percentage in strength reduction have been compiled. The compressive strength values have ranged from about 17 MPa to 42 MPa, while the percentage reductions in strength have ranged from 20-40%, again, for a wide range of water-cement ratios of the mix and different strengths of the original concretes. In one report [38], a reduction in strength of over 80% was observed. The percentage reduction in strength values for the case of recycled coarse and recycled fine aggregate concrete is about the same as for recycled coarse and virgin fine aggregate concretes (up to 40%); but, the attainable strengths are much lower. This is due to the additional detrimental effects that this aggregate fraction presents to the newly-made concrete. For these reports investigated, it is also possible to track these reductions in values over time in order to observe if any change or improvements have taken place in the use of recycled coarse and fine aggregate concrete. This is shown in Figure 3.4. It can again be seen again, from this chart, that there is no definite trend towards improvement in the technology of using recycled coarse and recycled fine aggregate concrete. This is not surprising considering that the fine fraction of the aggregates brings an additional degree of uncertainty to the quality of concrete to be expected.

Another interesting aspect with respect to compressive strength is the deviation from the conventional water-cement ratio law itself. Compressive strength is inversely related to the water-cement ratio of the mix; however, the relationship between these two variables changes for concretes made with normal aggregates and concretes made with recycled aggregates. From the same compilation of works as above [21, 22, 24, 26, 36, 37, 38], Figure 3.5 is plotted depicting the variation of compressive strength with water-cement ratio. It can be seen that recycled aggregates give a variation of concrete strengths

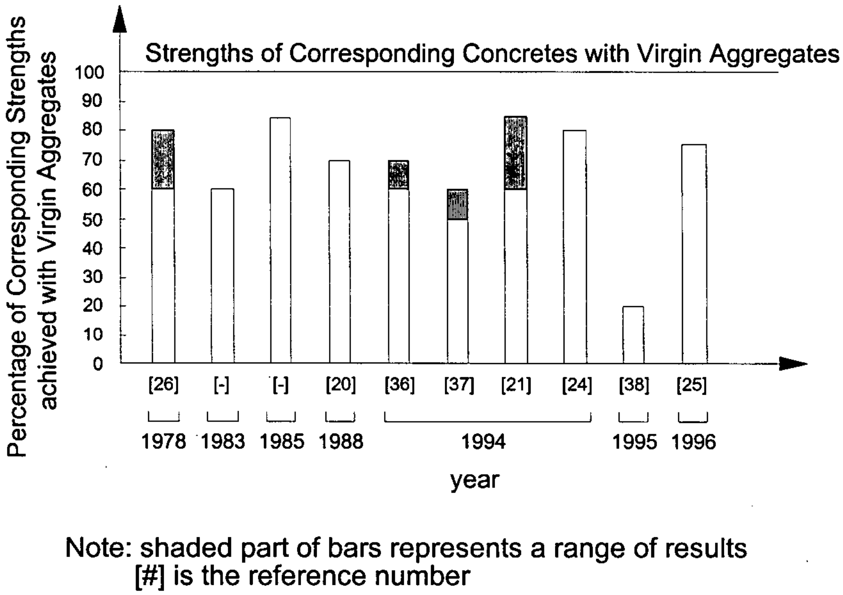


Figure 3.4 Strengths of recycled aggregate concrete as percentages of the strengths of corresponding virgin aggregate concrete over time (recycled coarse aggregates and recycled fine aggregates).

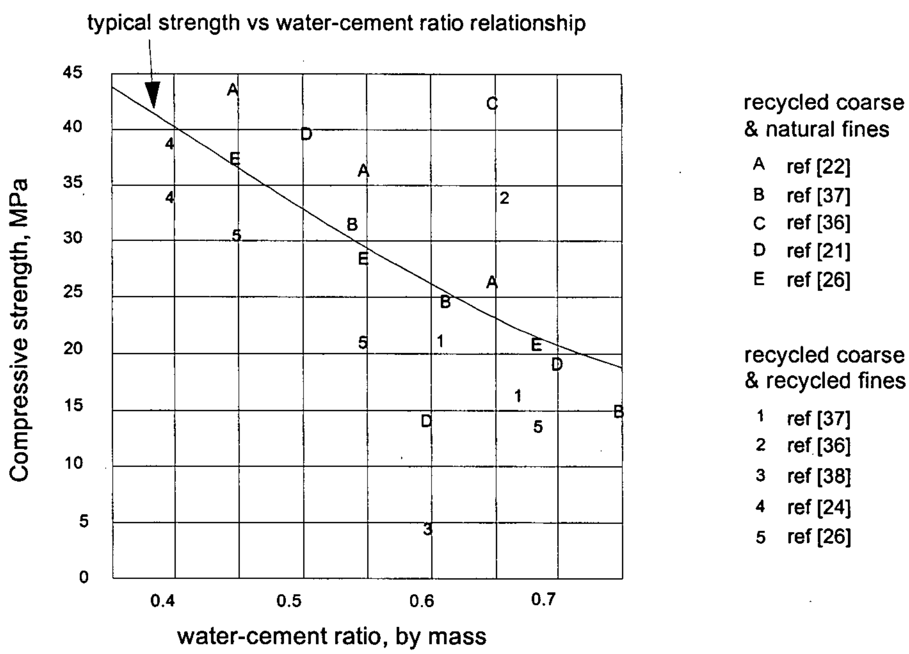


Figure 3.5 Variation of the compressive strengths of concrete with water-cement ratio for virgin and for recycled aggregate concretes.

comparable to those attained with normal aggregates. Concretes with the entire range of aggregate sizes substituted clearly show lower strengths than those with only the coarse aggregate fraction substituted. The general trend followed by these curves is similar in shape to that followed by the virgin aggregate concretes, although, due to the variability of the results, it is not certain whether the trends are exactly the same. Mukai et al. [39] found excellent straight line relationships between the cement-free water ratio and compressive strength, as shown in Figure 3.6, for recycled aggregate concrete made with coarse recycled aggregate and natural sand, as well as with coarse and fine recycled aggregate. Therefore, it was concluded that the water-cement ratio law generally applies to recycled aggregate concretes. However, from both Figures 3.5 and 3.6, it can be observed that the substitution of the entire aggregate fraction, coarse and fine, with recycled material produces lower strengths and a greater scatter of results. So, perhaps, one important consideration that should be kept in mind when producing recycled aggregate concrete is to screen off the fine recycled aggregate portion and use natural sand instead. This again supports Hansen's [10] suggestion to completely avoid the use of fine recycled aggregates.

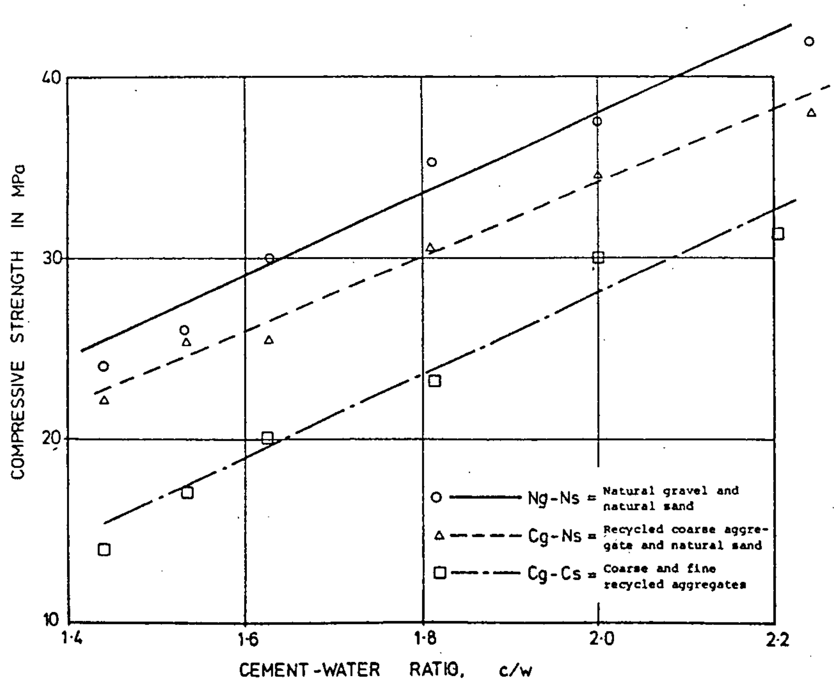


Figure 3.6 Relationship between cement-free water ratio and compressive strength [39].

As far as compressive strength gain is concerned, not much work has been done to investigate this behaviour. There are reports, however, that the rate of strength development in recycled aggregate concrete is very close to that in virgin aggregate concrete [36, 37]. This is seen from the curves following similar trends in the strength-age relationship. The results for one study are shown in Figure 3.7 [37].

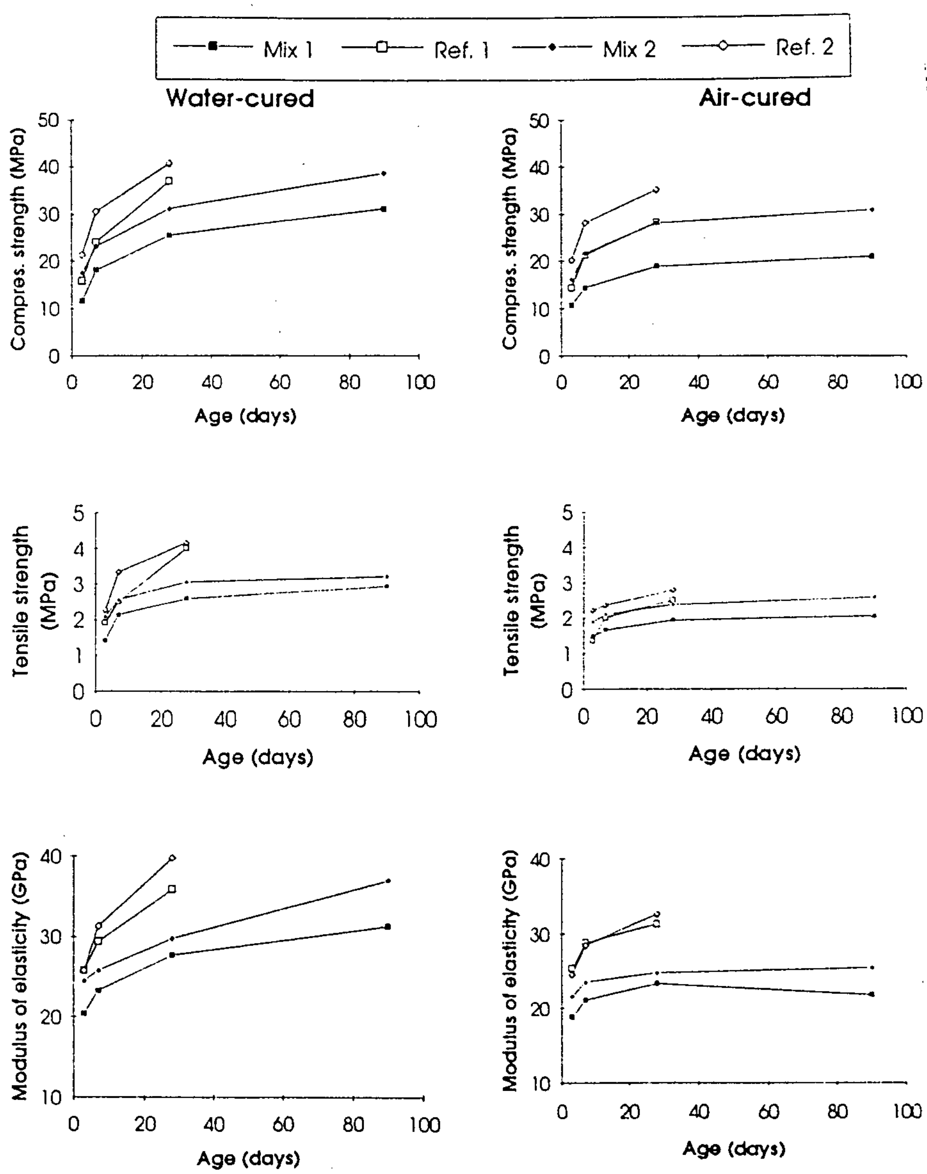


Figure 3.7 Compressive strength with age for recycled aggregate concrete (Mix 1 and 2) and natural aggregate concretes (Ref 1 and 2) [37].

3.5.2 Effect of Dry-Mixing of Aggregate

Dry-mixing of aggregates is a process that involves placing the aggregates in the mixer and allowing them to mix for a certain period of time prior to the addition of the other concrete making ingredients. This is done to remove the attached mortar from the surfaces of the aggregates, thereby leading to cleaner and better shaped particles. At the same time, weak, soft material can be broken up. Kasai et al. [40] investigated the effects of dry-mixing on the properties of recycled aggregate concrete. It is perhaps not surprising that the fineness modulus of recycled aggregates is reduced with increasing time of dry-mixing in the concrete mixer. This is because the attached mortar grinds off producing a higher percentage of fine particles. In the same investigations, they also found that the compressive strength, tensile strength, and modulus of elasticity of recycled aggregate concretes, made with recycled aggregates which had been dry-mixed prior to production of concrete, was considerably higher than the strengths and modulus of elasticity of corresponding concretes made with recycled aggregates which had not been dry-mixed prior to the addition of cement and water.

They suggested that the effects observed after dry-mixing may be due to one or more of the following reasons:

- the shape of the coarse aggregates is improved through dry-mixing leading to better workability and compaction;
- the old mortar which is attached to the surface of the recycled aggregate particles is partially removed by dry-mixing;
- the fine particles of old cement which are liberated during dry-mixing of recycled aggregates accelerate the hydration of fresh cement similar to chemical nucleating agents.

3.5.3 Modulus of Elasticity

The modulus of elasticity is the ratio of stress to strain in the elastic range of a stress-strain curve for concrete. Normal density concrete made with conventional aggregates has a modulus of elasticity somewhere between 14 GPa to 41 GPa, depending on factors such as compressive strength and aggregate type [41]. There are various expressions which relate this value to the compressive strength of concrete as a function of density [41, 73]. In recycled aggregate concretes, as for compressive strength, the modulus of elasticity has also been repeatedly reported to sustain a percentage reduction compared to a similarly proportioned concrete made with conventional aggregates. From the literature surveyed [10, 21, 25, 26, 37, 38], the majority of the studies report this percentage decrease in modulus of elasticity as opposed to the specific modulus value of the material. This is because the former value gives a better indication of the degree of loss in this property compared to that potentially achievable with conventional aggregates. These investigations have also considered replacing either just the coarse aggregate fraction or the complete aggregate phase.

The studies examined indicate that, when only the coarse fraction of aggregates is substituted with recycled material, the modulus of elasticity decreases by about 15-33%. On the other hand, when the entire aggregate range is replaced by recycled material, this percentage reduction is in the range of 20-40%. It is not surprising to see that, as with compressive strength, these values are affected more when the fine aggregate fraction is included in the mix. Also, it should come as no surprise that reductions in the modulus of elasticity lie in these given percentage ranges when recycled aggregates are used in concrete, considering that the compressive strength is affected at a slightly higher magnitude and these two material parameters are related through a square root relationship ($E \alpha (f_c')^{1/2}$). Thus, the values of elastic modulus for recycled aggregate concrete should lie in the range of 10 GPa to 35 GPa, which is what has been determined by some researchers [25, 37]. This reduction in modulus of elasticity is mainly attributed to the lower modulus of the attached mortar on the surface of the original aggregates [10].

Kakizaki et al. [42] developed an expression to deduce the minimum value for the modulus of elasticity of recycled aggregate concrete to be used in the design of structures made from such concrete, when the compressive strength, f_c, of the recycled aggregate concrete and the density, α, of the concrete are known. This is given as:

$$E_c = 2.1(10^5)*(\alpha/2.3)^{3/2}*(f_c/200)^{1/2}$$

This expression is based on observed elastic modulus values in the range of 25-40% lower than that of conventional concrete. However, there is insufficient data from the literature to verify this expression.

3.5.4 Stress-Strain Relationship

Limited data exists on the stress-strain relationship for recycled aggregate concrete. Some investigations have reported that the stress-strain relationship for recycled aggregate concrete is similar in shape to that of ordinary concrete. Thus, structures made from such material can be designed according to the theory of plasticity just like structures made from ordinary concrete [43]. Also, it has been found that the ultimate strain at compressive failure was $2.6(10^{-3})$ for recycled aggregate concrete made with both coarse and fine recycled aggregate, while it was $1.7(10^{-3})$ for both, an original control concrete and a concrete made with coarse recycled aggregate and fine natural aggregates [44].

3.5.5 Tensile and Flexural Strength

The tensile strength of concrete is normally measured as an indirect tensile strength from cylinder splitting tests or flexural tests. The values of the tensile strength obtained from such tests are higher than the true direct tensile strength, the splitting tensile strength being about 15% higher and the flexural strength (modulus of rupture) about 50% higher

[45]. However, they do provide an indication of the tensile resistance of the material as well as how it changes with varying material parameters.

Splitting cylinder test results on recycled aggregate concrete have shown varying results, from no difference to about a 20% decrease compared to virgin aggregate concrete. When concrete is made using coarse recycled aggregates and natural fines, the reported results on the percentage reduction have ranged from about 0-20% with actual strength values ranging from about 2.4 MPa to 4.5 MPa [10, 22, 24, 25, 26, 32, 37, 39]. On the other hand, when recycled fines are also included into the mix, some reports have found reductions of about 20% [26, 39], while others have found no differences [22, 24, 25]. Flexural strengths have also displayed varying results; but, the percentage drop seems to be even lower than that obtained with splitting tensile strength tests, at around 10% [10, 22, 24, 25, 26, 32]. It is also reported that the flexural strength of recycled aggregate concrete can be approximated as being somewhere between 1/5 and 1/8 of its compressive strength [26].

From the above, it seems that the indirect tensile strengths of recycled aggregate concretes do not vary as much as the compressive strengths. This appears surprising considering that tensile strengths should be more sensitive to aggregate variations than compressive strengths. It may be that not enough tests have been performed on this property to prove its sensitivity to aggregate variations and that the above reported values are within acceptable confidence intervals.

3.5.6 Drying Shrinkage and Creep

Drying shrinkage is the volumetric change that takes place in a mass of concrete when subjected to moisture changes, expanding with a gain of moisture and contracting with a loss of it. For normal concretes, it ranges from about $400(10^{-6})$ to $600(10^{-6})$ when placed in air at 50% relative humidity. Tests on recycled aggregate concrete have shown that drying shrinkage, either with only the coarse aggregate fraction replaced or with the entire aggregate fraction replaced, is much higher than concrete made with conventional aggregates [10, 21, 22, 32, 37, 46, 47, 48]. For concretes having their coarse aggregate part replaced by recycled material, the drying shrinkage is in the range of 10-50% higher than that of control specimens [10, 21, 22, 46, 47]. However, when the entire aggregate fraction is substituted with recycled material, the shrinkage values increase more and can reach up to 70-80% higher than those of control specimens [10, 21, 22, 32, 37, 46, 48]. To better illustrate this phenomenon, Figure 3.8 shows the evolution of drying shrinkage with time for both types of aggregate replacement. It is suggested that the greater drying shrinkage in recycled aggregate concrete is due to the fact that these concretes contain 50% more mortar than control concretes and drying shrinkage increases with an increase in the content of cement paste or mortar in concrete.

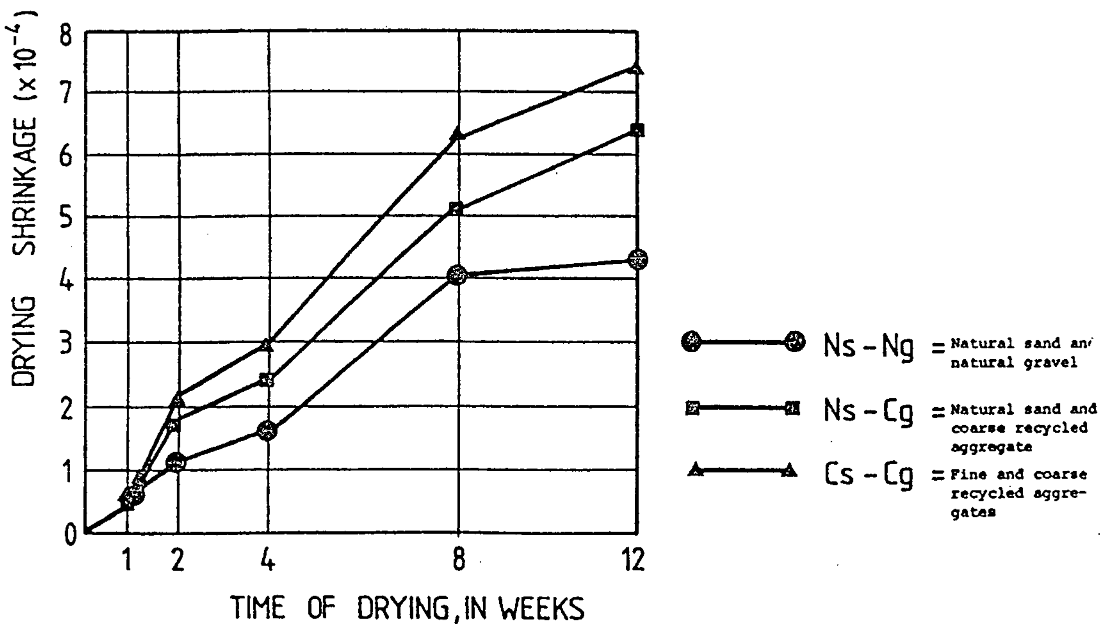


Figure 3.8 Drying shrinkage of original and recycled aggregate concretes as a function of drying time [46].

Creep in concrete is the time-dependent deformation that occurs under a sustained load as a result of movement of adsorbed water or interlayer gel water. Results on creep of recycled aggregate concrete show that this value is in the range of 25-60% higher than creep of normal aggregate concrete [10, 47, 48]. However, the difference in creep between concretes made with the two aggregate types remained almost constant at any water-cement ratio and at any sustained load level. This difference is probably due to the higher content of cement paste or mortar in recycled aggregate concretes.

3.5.7 Bond to Reinforcement

In a study by Mukai et al. [31], it was found that the bond strength between steel and recycled aggregate concrete is equivalent to that of conventional concrete under both static and fatigue loading, when coarse recycled aggregates were used with natural sand. However, when both fine and coarse recycled aggregates were used, cracks appeared at 15% lower flexural loads than when conventional aggregates were used, and the ultimate flexural strength of the reinforced concrete was 30% lower due to bond failure. It was also found that the shear strength of the reinforced concrete was slightly inferior to ordinary concretes but that shear strength could be increased with an increase in the number of reinforcing steel stirrups.

In a separate study by Yagishita et al. [49], it was found that, if recycled coarse aggregates are used for reinforced concretes which are dominantly subject to a flexural moment and uniaxial compression, the effects of the aggregate on the strength, ductility and mode of failure of the concrete member would be very slight. Only marginally larger cracks would form. However, when structural members are predominantly subjected to shear forces, cracks developed along the longitudinal reinforcement indicating a deteriorated bond to reinforcement when using recycled aggregates.

3.6 Durability of Recycled Aggregate Concrete

3.6.1 Permeability

Permeability is by far the most important property of concrete determining its durability. Apart from determining the relative ease with which the concrete can become saturated and vulnerable to frost damage, permeability also determines the possibility of a chemical agent penetrating into concrete and damaging the material, or attacking the reinforcement [50]. The permeability of mature, good quality concrete is approximately $1(10^{-10})$ cm/sec [41]. Since the overall permeability of the concrete depends on the permeability of its constituents, it is reasonable to expect that any changes in the constituents would have an effect on the overall permeability. Tests have, therefore, been carried out to determine the effects of the inclusion of recycled aggregates in concrete on its permeability.

In the various tests carried out, the water absorption of the concrete is measured in order to provide an idea of what the permeability is like. Any changes in the water absorption of the concrete would be indicative of changes in its permeability [26, 51]. As expected, for recycled aggregate concretes, the water absorption is about 2-5 times higher than corresponding concretes made with conventional aggregates [26]. However, the change in absorption from the control samples appears to depend in the relative strengths between the original aggregate and the new concrete [51]. If the recycled aggregate concrete is made to have a strength lower than the original aggregates, there appears to be no significant change in the water absorption from the control specimens. On the other hand, if the recycled aggregate concrete is made to have a strength higher than the original concrete, the absorption may be up to 3 times that of corresponding control samples.

Wainwright et al. [36] carried out tests to specifically determine the porosity and permeability of recycled aggregate concretes. They reported that, as with strength, the inclusion of recycled fines has a far greater adverse effect on porosity and permeability than the inclusion of recycled coarse aggregates alone. However, the presence of the recycled aggregate has a far greater effect on permeability and porosity than it does on strength. Permeability appears to be the most sensitive property, with values that could be as high as twice that of the control specimens when coarse and fine recycled aggregates are used.

3.6.2 Resistance to Freezing and Thawing

One of the most destructive weathering factors is freezing and thawing while the concrete is wet, particularly in the presence of deicing chemicals. Deterioration is caused by the freezing of the water in the paste, the aggregate particles, or both. The resistance to this is measured by the number of freeze/thaw cycles necessary to produce a certain amount of mass loss [41]. In the literature, experimental work on the freeze/thaw resistance of recycled aggregate concrete has shown somewhat contradictory results. Some authors have reported that the freeze/thaw resistance of such concretes is as good or better than that of corresponding control concretes [10, 21, 26, 32, 52]. On the other hand, other authors have reported that the frost resistance of recycled aggregate concretes is inferior, though just slightly, to corresponding control concretes [10]. These trends were seen regardless of whether only the coarse fraction of aggregates was replaced or the entire content of aggregates was replaced with recycled aggregates.

It appears that there are two factors working against each other which give rise to the variations in the results shown above. The higher porosity of the recycled aggregate concrete tends to be beneficial as it acts as a pressure reliever during the exertion of pressure due to osmotic or hydraulic differences while water freezes in the pores. However, the possibility of the original concrete from which the recycled aggregates was obtained being non-air entrained can lead to worsened freeze/thaw performance of the resulting new concrete. In other words, even if the recycled aggregate concrete is air entrained, it cannot be expected to be frost resistant unless the original concrete has also been air entrained. This is because the fracture surfaces can take place along the interface between cement mortars and original aggregate particles [10]. So, it has been recommended to not recycle concrete pavements which have been badly damaged by frost.

3.6.3 Alkali-Aggregate Reactions

Alkali-aggregate reactivity is a type of concrete deterioration due to a reaction between the active mineral constituents of some aggregates and the sodium and potassium alkaline components in the cement used [53, 54]. Three conditions are necessary to cause damaging alkali-aggregate reactivity in concrete. These are [55]:

1. Aggregates with enough reactive constituents that are soluble in the alkaline solutions;
2. Enough water-soluble alkali from some source (usually the cement) to drive the pH value of the pore solution in the concrete up to 14-15 and hold it there so that swelling alkali-silica gel is produced;
3. Enough water to maintain the solutions and provide moisture for the swelling of the gel.

The potential alkali-aggregate reactivity problem that may exist when recycled aggregates are used to produce concrete appears when the recycled material has suffered from previous similar reactions. These reactions may not have been completed and could continue in the new concrete. If petrographic or other examinations indicate that the reactive components have been used up, it may be safe to go ahead and use the material. If this is not the case, merely the use of a low-alkali cement in the new concrete may not prevent further alkali aggregate reaction with the recycled material. It is therefore necessary to perform long-term concrete prism expansion tests (CSA A23.2-14A) with the recycled material in cements with various alkali contents in order to determine the acceptable content of alkali [10]. Adding fly-ash may also prove to be helpful in preventing such reactions from occurring in the new concrete [55].

3.6.4 Carbonation and Reinforcement Corrosion

Carbonation of concrete is a process by which carbon dioxide from the air penetrates into the concrete and reacts with the hydroxides, such as calcium hydroxide, to form carbonates. In the reaction, calcium carbonate is formed; and this process increases the shrinkage on drying and lowers the alkalinity of the concrete, which leads to an increased potential for steel reinforcement corrosion [41]. In studies with recycled aggregate concrete, it was found that the rate of carbonation of a recycled aggregate concrete made with recycled aggregates from an original concrete which had already suffered carbonation was 65% higher than that of a control concrete made with conventional aggregates [26, 51]. Carbonation leads to a faster corrosion of the reinforcement in recycled aggregate concretes. However, such increased risk of corrosion can be offset by lowering the water-cement ratio of the concrete to be produced.

3.7 Shotcrete

3.7.1 Definition and Applications

Shotcrete can be defined as mortar or concrete that is pneumatically projected onto a surface at high velocity. Dry-mix shotcrete was first developed in 1911, but the principles have remained unchanged. The relatively dry mixture is consolidated by the impacting force and can be placed on vertical or horizontal surfaces without sagging. At the same time, because of the manner in which it is applied, it can be used in places where the use of formwork is impractical, where access to equipment is restricted, where thin layers and/or variable thickness are required, or where normal casting techniques cannot be employed. As a result, shotcrete can be used in many different fields of application and the number of potential uses for it continues increasing.

Shotcrete can be used instead of conventional concrete in most instances, the choice based on convenience and cost. It offers advantages over conventional concrete in a variety of new construction and repair work. There are three general categories of shotcrete applications. These are [56]:

1. Conventional Shotcrete
   - for construction of new structures;
   - for linings and coatings over existing structures;
   - for repair of deteriorated concrete;
   - for strengthening and reinforcing concrete elements.
2. Refractory Shotcrete

   used extensively in the new construction and in the repair and maintenance of steel, nonferrous metal, chemical, mineral and ceramic processing plants, steam power generation, and incinerators.
3. Special Shotcrete

   includes proprietary mixtures for corrosion and chemical resistant coatings in highly aggressive environments.

As the number of potential applications increases, there are more opportunities for the development of specialized shotcretes. These include the addition of mineral admixtures such as fly ash, silica fume or high-reactivity metakaolin to improve the fresh and hardened shotcrete properties or the addition of accelerators to improve the efficiency of the process. Another significant development is the use of fiber reinforcement in shotcrete to provide improved shrinkage resistance and, more importantly, enhance its toughness.

3.7.2 Processes

There are two very distinct types of shotcreting processes: the dry-mix process and the wet-mix process. Due to the dissimilarities in the production method between these two processes and in the water requirements, the resulting hardened materials are fundamentally different.

In the dry-mix process, cement, aggregates and possibly dry additives are batched, mixed and fed into the delivery equipment called the gun. The gun meters the dry materials into a pressurized air stream and delivers the mix through a hose to a nozzle. At the nozzle, water is injected through a water ring and the wet shotcrete is projected into place. The amount of water can be controlled by the nozzleman. A schematic arrangement of the dry-mix process is shown in Figure 3.9. These machines have an output of about 0.5-4 m^3/hr, and the particles are ejected at velocities of about 30-60 m/s [57].

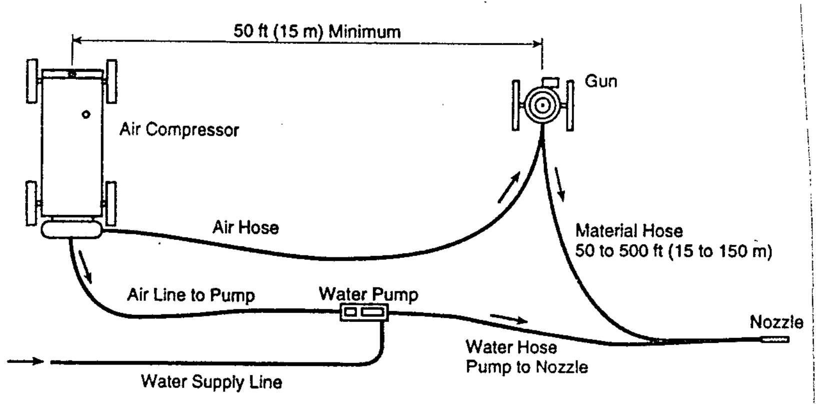


Figure 3.9 Schematic arrangement of the dry-mix process shotcrete [57].

The plastic properties of dry-mix shotcrete are determined by the proportions of the dry materials in the mixture, the amount of water added at the nozzle and the nozzling technique. The method of water addition is crucial to this production process because sufficient water must be added to obtain good compaction and reinforcement encasing, but not excessive water, which may cause sloughing or sagging of the concrete away from the substrate. Within this range, decreasing the water content will generally improve strength and other hardened properties. However, it also increases the rebound of material, which can be significant. An experienced nozzleman is crucial in this case to determine the optimal water to be added and to assure, through proper nozzle manipulation, complete mixing of the material as it strikes the receiving surface. Silica fume and/or accelerators have also been extensively used to achieve a much greater thickness of build-up and to reduce the amount of rebound (material and fiber rebound).

In the wet-mix process, cement, aggregates, water and often additives are batched, mixed, and fed into the delivery equipment (a pump or gun). There are two types of wet-mix process delivery: hydraulic and pneumatic. In the hydraulic method, the wet concrete is placed in a concrete pump and pumped through the delivery hose to the nozzle where air is injected to shoot out the wet mix (Figure 3.10). Particle velocities of 10-30 m/s are normally produced, while the spraying rates reach a maximum of about 20 m^3/hr. In the pneumatic method, the wet-mix operating principle is similar to the dry process, except that compressed air assists gravity in discharging each rotor chamber into the delivery line where further air is added to convey the wet material to the nozzle. Output rates are normally about 10-20 m^3/hr.

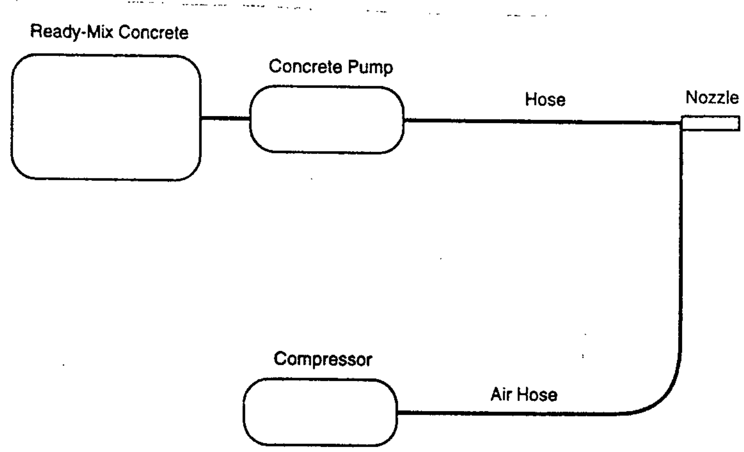


Figure 3.10 Schematic arrangement of the wet-mix process shotcrete (hydraulic) [57].

The plastic properties of wet-mix shotcrete are largely determined by the quality of the mixture supplied to the delivery equipment. If the mix is too wet, the nozzleman will have difficulties in making the shotcrete adhere to the substrate; however, if the mix is too dry, there could be difficulties in pumping and in consolidation. Normally, the slump is kept in the range of 40-80 mm. Silica fume can also be applied and the effect of this is to increase the build-up thickness, although this beneficial effect is not as pronounced as it is in the dry-mix process. Generally, material and fiber rebound in the wet-mix process shotcrete is lower than in the dry-mix process; but, wet-mix shotcrete usually has a higher water-cement ratio than dry-mix shotcrete, unless superplasticizers are used, in which case lower water-cement ratios are attainable.

Shotcrete suitable for normal construction requirements can be produced by either process. However, differences in capital and maintenance cost of equipment, operational features, suitability of available aggregate, and placement characteristics may make one or the other more attractive for a particular application. Table 3.1 compares the two processes [56].

Table 3.1 Comparison of dry- and wet-mix processes [56].

Dry-mix process	Wet-mix process
1. Instantaneous control over mixing water and consistency of mix at the nozzle to meet variable field conditions.	1. Mixing water is controlled at the delivery equipment and can be accurately measured.
2. Better suited for placing mixes containing lightweight aggregates, refractory materials and shotcrete requiring early strength properties.	2. Better assurance that the mixing water is thoroughly mixed with other ingredients.
3. Start and stop placement characteristics are better with minimal waste and greater placement flexibility.	3. Less dusting and cement loss accompanies the gunning operation.
	4. Normally has lower rebound (material and fiber) resulting in less material waste.
	5. Capable of greater production.

3.7.3 Properties

In general, properly applied shotcrete is a structurally sound and durable construction material and exhibits excellent bonding characteristics with concrete, masonry, rock, steel, and many other materials. The physical properties of shotcrete are comparable or superior to those of conventional mortar or concrete having the same composition. The compressive strength, as measured from drilled cores, typically lies in the range of 30-60 MPa for dry-mix shotcrete with aggregate-cement ratios between 4:1 and 2.5:1 and unspecified water-cement ratios; while, for wet-mix shotcrete, it is in the range of 20-45 MPa for water-cement ratios of 0.7 to 0.45. Silica fume and superplasticizer additions can increase the compressive strength to values up to 80 MPa in the wet process. The addition of fibers into shotcrete increases the compressive strength by only a marginal value; so, it is not recommended to add fibers just for compressive strength improvement reasons. The rate of strength gain is also an important factor as it has important significance for applications like tunneling where early strengths are required. Early age strength can be quickened by the use of accelerators, although long-term properties might also be adversely affected. The flexural strengths, measured from standard beams, are similar to cast concrete flexural strengths, in the range of 3-5 MPa. When silica fume is added, this strength also increases. With the addition of steel fibers, due to the preferred orientation parallel to the surface being sprayed, the ultimate flexural strength is improved at higher fiber volumes. The flexural toughness is the property that is most significantly improved by the additions of fibers. Although there are discrepancies in the various methods by which this property is determined, in general, marked improvements in the post-peak load carrying capacity and energy absorption are obtained when fibers of suitable geometry are incorporated into shotcrete. The impact resistance of shotcrete is another important property of this material as shotcrete is frequently used in applications which could be subject to violent loading conditions, such as rockbursts. The addition of fibers has an extremely beneficial effect on this property as it enhances its energy absorbing capacities. As far as shrinkage is concerned, shotcrete is slightly more prone to this phenomenon than normal concrete since it has higher cement contents and less coarse aggregates; therefore, this may require a closer control joint spacing or increased use of reinforcement. Properly applied shotcrete is a relatively dense, well compacted material and consequently, its water absorption is quite low. Thus, the ASTM C642 test for determination of boiled absorption and volume of permeable voids is quite commonly used in sprayed concrete specifications and quality control testing. These two parameters can be used as indicators of shotcrete quality.

There has been much debate regarding the prediction of shotcrete properties by extrapolating the corresponding properties from cast concrete. As a first approximation, this might appear feasible. However, when one considers the method of application and compaction, the mode of interaction between the cement, the water and the aggregates, and the different rheology of the mix, it is implied that the kinetics and strength gain properties are expected to be different from those in cast concrete. In an experiment carried out by Banthia et al. [58], fiber reinforced specimens were both shot and cast, and tested for compressive strength, and flexural strength and toughness. It was found that the compressive strength of shotcrete is significantly lower than its cast counterpart; however, the flexural strength is higher. At the same time, the correlation between the flexural toughness of shotcrete and its cast counterpart appears to depend on the specific fiber geometry used. Therefore, it was concluded that there are risks in trying to predict shotcrete behaviour on the basis of the behaviour of similarly proportioned cast concrete.

4.0 CHARACTERIZATION OF RECYCLED AGGREGATES USED

4.1 Source

The recycled aggregates used for the research experiments were obtained from a local concrete recycler called Columbia Bitulithic Ltd. located in Richmond, British Columbia. The sources of the concrete wastes are from structure demolition projects, structural renovations, sidewalk removals, and roadworks from the surrounding municipalities mainly, the cities of Vancouver and Richmond. These wastes arrive at the plant on a daily basis with annual quantities of about 200,000 tons. Since many of these wastes are part of demolition projects, other construction wastes like wood, drywall, rubber, and glass may also be present in the concrete waste stream. Concrete with other toxic materials like leachates, asbestos or oil is not accepted into the plant.

The concrete wastes received at Columbia Bitulithic Ltd. are in the form of chunks of about 1 m in size and down. These are ground down in three stages through the following series of crushers: jaw crusher, rotary crusher and cone crusher. Figure 4.1 shows the line of crushers at Columbia Bitulithic Ltd. Contaminants are removed as far as possible through magnetic separators, hand picking and air classifiers, but the resulting material is never completely free from them. Coming out of the primary crusher is a material of about 150-200 mm in size, and the final product is a material of size 20 mm minus. The crushers are adjusted to specified openings that produce materials satisfying the gradation limits for a given application, mainly road subbase. Normally, if the gradations are still not quite within limits, natural sand is blended into the recycled aggregates to bring the material to within the appropriate gradation limits. The resulting product is sold at $7.75 per metric ton (in 1998) to the surrounding municipalities mainly for use as a fill material and for road building.

Figure 4.1 A view of the system of concrete crushers at Columbia Bitulithic Ltd.

4.2 Characteristics and Properties

Aggregates generally comprise over 70-80% of the volume in concrete. Many of the properties that aggregates possess, either physical or chemical, therefore have a large influence on the properties of the fresh concrete mixture, as well as on the properties of the hardened concrete. In order that a concrete be strong, aggregates should be hard and strong; free of soft, porous materials; and free of fine impurities and organic material which could interfere with the cement-aggregate bond. In order that a concrete be durable, aggregates should be resistant to weathering action; free of any minerals which could lead to adverse chemical reactions; and free of impurities which could affect the strength and soundness of the cement paste. There are many aggregate sources in nature that provide material with these characteristics; and, standard tests can be performed to evaluate their suitability. Besides the properties required for strong and durable concrete, there are other aggregate properties required for mix proportioning of suitable concrete mixes. These are: (1) shape and texture, (2) size and gradation, (3) bulk density, (4) free moisture content and absorption, and (5) specific gravity. It is important to recognize the properties that a given batch of aggregates possesses in order to know what possible effects these may have on the resulting concrete mixture and what measures to take in order to achieve desirable concrete properties.

Recycling crushed waste concrete as aggregates must also take into account the requirements for conventional aggregates for their use in concrete. Due to the origin of waste concrete and the way in which it is processed, a number of differences exist between the properties and characteristics of recycled aggregates and virgin aggregates. Many of these different properties must be recognized and measures taken to minimize possible adverse effects, in order to produce concretes with satisfactory fresh and hardened properties. The following provides a description of the properties of the aggregates hauled from Columbia Bitulithic Ltd. and the significance that these may have on the resulting concrete. These properties mainly relate to the workability of the fresh concrete mix and the strength of the hardened concrete. In a complete evaluation of aggregates, tests for durability must also be performed. Physical durability tests include abrasion and frost resistance, while chemical durability tests include evaluation of the potential for alkali-aggregate reactions and the presence of reactive substances like sulphates. These durability tests were, however, not carried out as part of the research. Properties of recycled aggregates investigated, except for size gradation which will be discussed in the next section, are described as follows.

1. Shape and Texture

   Figure 4.2 shows a photograph of the recycled aggregates that were obtained from the concrete recycling plant. They appear as any normal batch of aggregates would look like, except for an extra dustiness. Because of the crushing operations, most of the coarse particles tend to be angular with many sharp edges and corners.

Figure 4.2 Recycled aggregates from Columbia Bitulithic Ltd.

Figure 4.3 Particles with and without attached mortar as taken from recycled aggregates.

At the same time, due to the attached mortar (Figure 4.3), the fractured surfaces and the presence of other substances, the surface of these particles tend to be rough in nature. Such angular and rough textured particles have a higher surface-to-volume ratio, therefore requiring more paste to fully coat the surfaces and leading to greater interparticle interactions during mixing and handling. The result is a less workable mix. However, these types of particles can improve mechanical properties through better interlocking, as long as the particles are strong enough to resist the interlocking stresses.

2. Bulk Density

   The bulk density of the aggregates is required when batching is done on a volumetric basis. The procedure for determining this is outlined in ASTM C29 [59]. As obtained from the recycling plant, the recycled aggregates have a dry-rodded bulk density of 1720 kg/m^3 which is in the range of other normally used aggregates.

3. Moisture Content

   The moisture content represents the amount of water the aggregates contain in their bulk state. As outlined in ASTM C566 [60], the moisture content of the aggregates, as hauled from the recycling plant, is about 6.0-6.5%. This value could vary according to exposure conditions.

4. Absorption

   The absorption represents the maximum amount of water the aggregates can absorb. It can be used in mix proportioning calculations to convert from the SSD (saturated-surface dried) to the OD (oven-dried) state, or vice versa. One of the main features of recycled aggregates is its much higher water absorption compared to virgin ones. This comes as a result of the presence of adhered mortar on the aggregate surfaces, as well as the presence of other more absorptive substances. CSA A23.2-6A, A23.2-11A and A23.2-12A present standards for determination of the absorption of coarse and fine aggregates. As mentioned in Section 3.3.3, the determination of water absorption for fine recycled aggregates is not straightforward, due to the difficulty in estimating the SSD condition because of the cohesiveness of the particles. So, judgement has to be used to assess this condition.

Following the standard methods, the water absorption of coarse recycled aggregates (> 5 mm) was found to be 3.7% while that of fine recycled aggregates (< 5 mm) was found to be as high as 12%. This is definitely much higher than corresponding values for virgin aggregates which are in the range of 0.5-1.7% for coarse and fine material. The absorption values were also determined for aggregates that had been dry-mixed for 25 min. Dry-mixing slightly decreased the water absorption of coarse recycled aggregates while there was no difference for fine recycled aggregates. It is believed that, although dry-mixing helps remove the attached mortar from the surfaces of coarse particles, the resulting dust remains on the particles, leading to equally high absorptions.

The effective absorptions of the coarse and fine aggregates were also determined. These values are necessary to calculate the amount of water that would be removed from the concrete mix if they are not in the SSD condition. The values were obtained from 30 min. absorption tests and they are deemed to be representative of the amount of water the aggregates will absorb from the mix before it has hydrated enough to prevent any more water being removed from it. Effective absorption values were found to be 0.73% for coarse recycled aggregates and 8.6% for fine recycled aggregates. Given the proportions that these two fractions exist in the batch of aggregates, the overall effective absorption is 5.1%.

5. Relative Density

Relative densities can be distinguished between an apparent relative density and a bulk relative density. The former refers only to the solid material excluding the pores, whereas the latter refers to the entire volume of material, including both, the permeable and impermeable voids. For purposes of mix proportioning, to establish mass-volume relationships, the bulk relative density is utilized as it represents the effective volume that aggregates occupy. Specific gravity values were only determined for the coarse aggregate fraction. For the recycled aggregates, the bulk relative density was found to be 2.48. This value is quite comparable although slightly lower than that of virgin coarse aggregates which is usually in the range of 2.5-2.8.

6. <u>Contaminants Present</u>

Since the recycled aggregates were obtained from crushed demolition wastes, it is not surprising to expect a substantial amount of foreign substances, or contaminants. Basically, these contaminants can be defined as any foreign substance other than the original gravel or sand and the majority of these are clearly visible from the mix of aggregates. From a visual inspection of the aggregates hauled, the contaminants seen in the concrete, in descending order, are as follows:

a) wood particles and chips which appeared as very fibrous material,
b) asphalt chunks which dissolved upon heating,
c) brick chunks which were visible as red particles,
d) cardboard pieces,
e) glass fragments,
f) small gypsum pieces,
g) plastic chunks which were probably PVC,
h) scraps of metal.

From the above descriptions, it can be seen that recycled aggregates, although also originating from concrete, possess properties which could significantly differ from those of virgin aggregates. Many of these discrepancies have to be compensated for in some way to achieve results in concrete made from these aggregates that would be similar to those obtained if virgin material was used. For example, the larger absorption would call for a higher water content to be added into the mix for a given workability. The same can be said about the particle shape and texture. Therefore, batching procedures would have to be modified to account for these different properties. Contaminants cannot be readily separated from waste concrete. It would be very expensive to have extremely selective procedures that could separate the contaminants from the waste concrete stream. As a result, it is necessary to understand the potential effects these materials may have on the expected performance of the concrete and to evaluate whether or not it is acceptable to have such effects. If not, one should also consider possible methods to mitigate the adverse influences of the contaminants.

4.3 Gradation

The gradation or particle size distribution of a given batch of aggregates is an important characteristic because it determines many of the properties that the aggregates will impart to the final product. It can also potentially influence the economy of such a use. Gradation is usually expressed as the cumulative percent passing (finer) or cumulative percent retained (coarser) on each of a series of standard sizes and is presented as gradation charts. The crushed waste concrete at Columbia Bitulithic Ltd. has its gradation changed from time to time depending on the job for which it is required. This can easily be done by changing or adjusting the screens within the line of crushers and by adding different blends of sand to the stream. At the time the recycled aggregates were obtained (August 1997), they had the gradation shown in Table 4.1. The nominal maximum aggregate size is specified as being 19 mm; and, the overall fineness modulus, obtained from the cumulative percent retained on standard sizes, is 4.6. The fineness modulus for the sand sizes is 2.8.

Table 4.1 Gradation of recycled aggregates as hauled from Columbia Bitulithic Ltd.

sieve size	Cumulative percent passing
20 mm	99
14 mm	82
10 mm	70
5 mm	53
2.5 mm	43
1.25 mm	34
630 μm	24
315 μm	12
160 μm	6
80 μm	3

In concrete applications, the aggregate gradation plays an important role since only suitable gradations can secure workability and economy in the use of cement. Cement is generally the most expensive ingredient in concrete; therefore, the use of a wide range of aggregate sizes from coarse to fine in such a way to create dense packing by reducing the void space produces the most economical mix. However, such dense particle size distributions do not give a workable concrete. Thus, a compromise has to be worked out between workability and economy. Based on practical experience on a number of jobs, ASTM C33 [19] sets grading limits for aggregates to be used in concrete. These limits are intended so that suitable aggregate gradations can be obtained in order to achieve an acceptable trade-off between the resulting concrete properties and the overall cost. If an aggregate does not conform to these limits, it does not necessarily mean that concrete cannot be made with the aggregate but that the concrete may require more paste and may be more liable to segregation during handling and placing. Fine and coarse aggregates have separate grading limits, the fine gradation limits being a general one and the coarse gradation limits being dependent on the range of coarse sizes in question. If the recycled aggregates is to be used in concrete applications, its gradation should satisfy the ASTM C33 requirements. The recycled aggregate gradation, as shown previously in Table 4.1, is for the entire range of sizes; but, it can be easily separated into the individual gradations for the fine part and for the coarse part by considering that the entire range of aggregate sizes consists of a blend of two components: a fine one (anything smaller than the No.4 sieve) and a coarse one (anything larger than the No.4 sieve). The individual gradations for the fine and the coarse fractions of the recycled aggregates, along with the stipulated limits, are shown in Table 4.2. This is graphically shown in Figure 4.4.

Table 4.2 Gradations of fine and coarse fractions of the recycled aggregates.

Fine Fraction (Cumulative percent passing)			Coarse Fraction (Cumulative percent passing)		
sieve size	recycled aggregates	CSA A23.1 limits	sieve size	recycled aggregates	CSA A23.1 limits
10 mm	100	100	25 mm	100	100
5 mm	100	95-100	20 mm	98	90-100
2.5 mm	81	80-100	14 mm	62	-
1.25 mm	64	50-90	10 mm	36	20-60
630 μm	45	25-65	5 mm	0	0-10
315 μm	23	10-35	2.5 mm	0	0-5
160 μm	11	2-10			

Fineness Modulus = 2.8

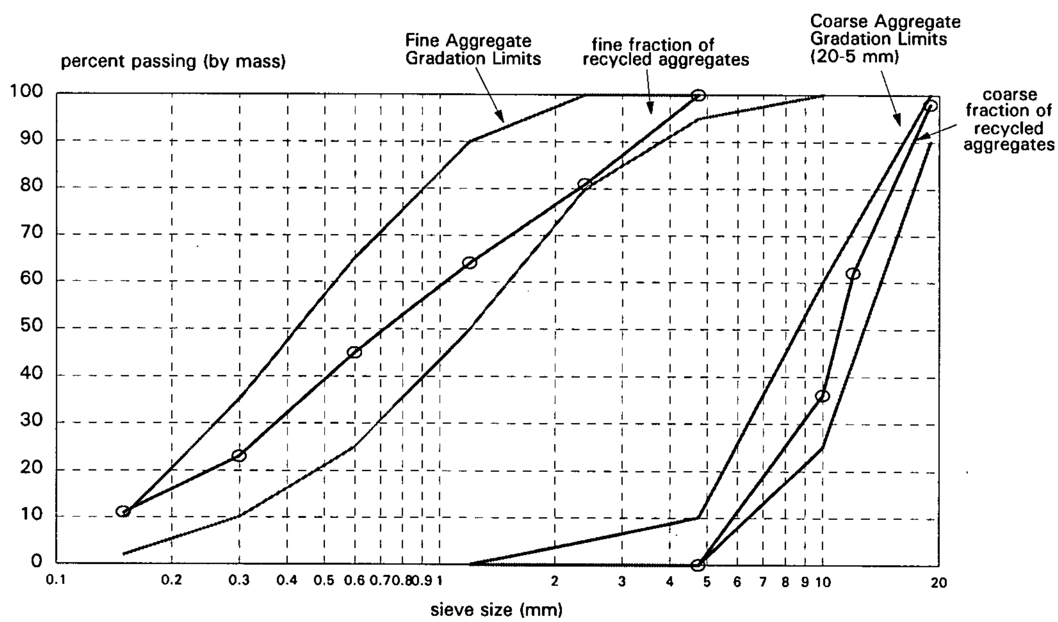


Figure 4.4 Gradations of fine and coarse fractions of the recycled aggregates compared to the CSA A23.1 limits for concrete aggregates.

It can be seen from the table or the chart that the fine and the coarse portions of the recycled aggregates obtained from the recyclers are well within the required limits. Besides satisfying these limits, there are other requirements for fine aggregates which are also satisfied, such as having no more than 45% of material retained between any two consecutive sieves and that the fineness modulus be between 2.3 and 3.1. The amount of material passing the No.200 sieve for normal fine aggregates is limited to 5% by the ASTM, but the recycled aggregates contain 6% of such sizes in the fine part. This may give rise to a higher water demand and some volume instability of the concrete in the long term. Other than this, in general, from a gradation point of view, the recycled aggregates are satisfactory for concrete applications and are expected to produce concretes displaying good workability properties.

In shotcrete applications, the grading of aggregates is more critical due to the lack of external vibration and the changes in mix proportions as a result of rebound (particularly in the dry-mix process). The heavier coarser aggregate particles rebound more than the finer ones, resulting in a more finely graded material in-situ with a higher cement content than the as-batched material. From a processability point of view, aggregate gradation is more crucial in the wet-mix process than in the dry-mix process because, in wet-mix shotcrete, there is need to achieve a balance between the concrete characteristics required to produce a pumpable mix and those characteristics required to project the shotcrete into place with minimum rebound losses and segregation. Also from practical experience, gradation limits have been stipulated by the ACI Committee 506 [56] for aggregates to be used in shotcrete. There are three different limits: Gradation 1 which is essentially a sand gradation used in the dry-mix process; Gradation 2 which is coarser and can be used for both wet-mix and dry-mix; and Gradation 3 which is occasionally used for wet-mix shotcrete. If recycled aggregates are to be used in shotcrete applications, the gradation should satisfy one of the ACI 506 bounds, depending on the particular application at hand. The recycled aggregate gradation, along with the ACI 506 stipulated limits, are shown in Table 4.3 and graphically in Figure 4.5. Since the ACI 506 uses imperial sieve numbers, the gradation of the aggregates will be expressed in terms of these. Their corresponding standard sizes are as follows: 3/4" = 20 mm, 1/2" = 12 mm, 3/8" = 10 mm, No.4 = 4.75 mm, No.8 = 2.4 mm, No.16 = 1.25 mm, No.30 = 600 μm, No.50 = 300 μm, No.100 = 150 μm, and No.200 = 75 μm.

Table 4.3 Gradation of recycled aggregates compared to ACI 506 limits.

	Cumulative percent passing			
sieve number	recycled aggregates	Gradation 1 Limits	Gradation 2 limits	Gradation 3 limits
3/4"	99	-	-	100
1/2"	82	-	100	80-95
3/8"	70	100	90-100	70-90
No.4	53	95-100	70-85	50-70
No.8	43	80-100	50-70	35-55
No.16	34	50-85	35-55	20-40
No.30	24	25-60	20-35	10-30
No.50	12	10-30	8-20	5-17
No.100	6	2-10	2-10	2-10

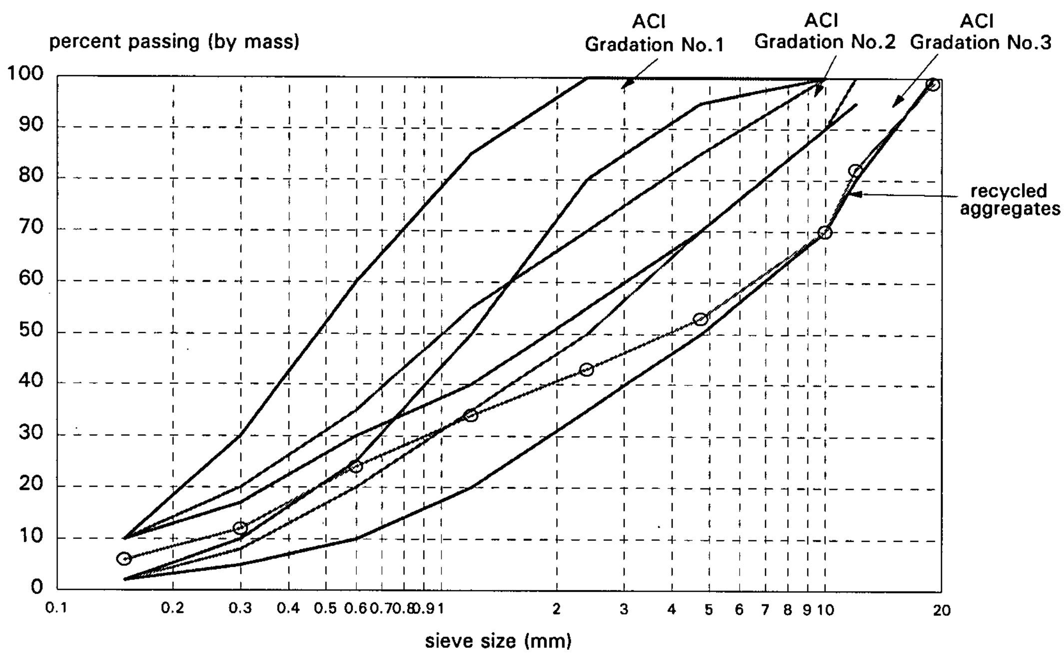


Figure 4.5 Gradations of recycled aggregates compared to the ACI 506 limits for shotcrete aggregates.

Again, it can be seen that the recycled aggregates, as obtained from the recyclers, fall within the ACI 506 Gradation No.3 limits. In other words, these aggregates are suitable for use in wet-mix shotcrete from a gradation perspective. If they were to be used for dry-mix shotcrete, the size fractions above the 3/8" sieve (10 mm) would have to be removed in order to meet Gradation 2 and the fractions above the No.4 sieve (5 mm) would have to be taken out to meet Gradation 1. This can be done without a problem by passing the material through screens of that size to remove anything coarser than the screen size. Therefore, again, from a gradation point of view, the recycled aggregates can be satisfactorily employed in shotcrete. In practice, most shotcrete is done with Gradation No.2 and dry-mix shotcrete done with Gradation No.1; but, for the purposes of the research, wet-mix shotcrete was done with Gradation No.3 limits while dry-mix shotcrete was done with Gradation No.2.

4.4 Effects of Dry-Mixing

In various investigations, the effect of dry-mixing the recycled aggregates has been examined. This involves placing the material in its dry state into a mixer and allowing the mixer to run for a certain period of time. Through the interparticle grinding and shearing, the attached mortar from the surfaces of the particles is removed thus, cleaning the original aggregates from this adhered material, which is known to create a number of deleterious effects. Recycled aggregates that have been dry-mixed have shown lesser percentage reductions in strength (compressive and tensile) and in elastic modulus when compared to similar aggregates that have not been dry-mixed [40]. These improvements have been attributed to the following:

- The shape of the coarse aggregates is smoothened through dry-mixing;
- The old mortar which is attached to the surface of the recycled aggregate particles is removed by dry-mixing;
- Soft, weak, porous or friable particles are broken up and disintegrated during dry-mixing;

- The fine particles of old cement which are liberated during dry-mixing of recycled aggregates accelerate the hydration of fresh cement in a similar way to chemical nucleating agents [40].

For these reasons, it appears that the dry-mixing process is particularly beneficial for coarse aggregates; and, since coarse aggregates are far more influential on strength than fine aggregates are, it is not surprising to see that such a process actually results in strength improvements of the resulting concretes. This is because a better bond can be developed between the new matrix and the original coarse aggregate thereby, utilizing more of the potential strength and properties that these original coarse aggregates have. Also, this process can get rid of the weak material which would be the most probable locations for failure initiation under loading conditions. Materials in this category would include the soft, porous and friable products such as brick, gypsum and old mortar. Moreover, if the fine particles liberated are of extreme fineness, besides acting as possible nucleating agents, they can also act as fillers in the matrix and in the matrix-aggregate transition zone, further improving the strength of the resulting concrete [61]. On the other hand, due to the amount of fine powder produced, the aggregate surfaces can become covered with this dust which could prove to be detrimental to bonding. So, it appears that a number of factors are present during the dry-mixing of the aggregates for which the end result is a slight improvement in mechanical properties as has been observed by several researchers.

The recycled aggregates hauled from Columbia Bitulithic Ltd. were dry-mixed and the effects of this process were assessed after specific time intervals throughout the dry-mixing process in order to track the changes that occurred. The total dry-mixing time was 25 min, at which time no appreciable changes occurred from the previous recording time. Recycled aggregates were placed in a pan mixer, and at total dry-mixing times of 3 min, 6 min, 10 min, 15 min, and 25 min, the aggregates were assessed for the following characteristics:

- visual observation of particles appearance and condition;
- gradation and fineness modulus.

The visual observations were as follows:

- <u>0 min</u>
  - start of dry-mixing process; aggregates not yet mixed.
  - appearance is normal.

- <u>3 min</u>
  - more fine powdery material present.
  - more material in the 5-10 mm size range present.
  - still a number of large particles.

- <u>6 min</u>
  - more fine material, much of it covering the large particles.
  - still lots of material in the 5-10 mm range.
  - mix turning brownish in colour.

- <u>10 min</u>
  - observations similar to those at 6 min except for more dust.
  - more particles in the 5 mm size.
  - mixing still very loud meaning that there are still lots of interparticle collision.

- <u>15 min</u>
  - a number of the large particles are no longer visible, except for a few particles with fairly clean surfaces.
  - lots of dust in the pan and in the air.
  - colour of mix has completely changed to brown.
  - noise from mixing gradually reducing.

- <u>25 min</u>
  - most large size particles over 20 mm broken up; large sizes mainly in the range of 10-20 mm.
  - similar in appearance to that at 15 min, except for more dust and a clayey feeling when touched.
  - almost all coarser particles, down to even the No.16 sieve, had no old mortar stuck on the surface, except for hidden areas on the aggregate.
  - mix is completely brown.
  - could hardly see the other particles like brick or asphalt.
  - mixing noise reduced.

- dry-mixing process stopped.

In effect, what appears to have occurred during the dry-mixing process is the stripping and pulverization of the adhered mortar from the original aggregate surface. This, in addition to the breakage of weak contaminants, produced a large amount of dust in the mix and a change in the colour. After a certain point into the process, the aggregates might have attained better shapes and the dust smoothened the motion of aggregates past each other hence, gradually reducing the noise produced from the mixing. Figure 4.6 shows the recycled aggregates before and after dry-mixing. Note the increased amount of dust and the change in colour. Because of the increased amount of dust, the interparticle action seems to have ceased after a certain point of time implying that, for a given type of mixer and a given size of aggregate batch, dry-mixing is effective only to a certain point. It is also interesting to note that the size of the largest particles remained unchanged. In

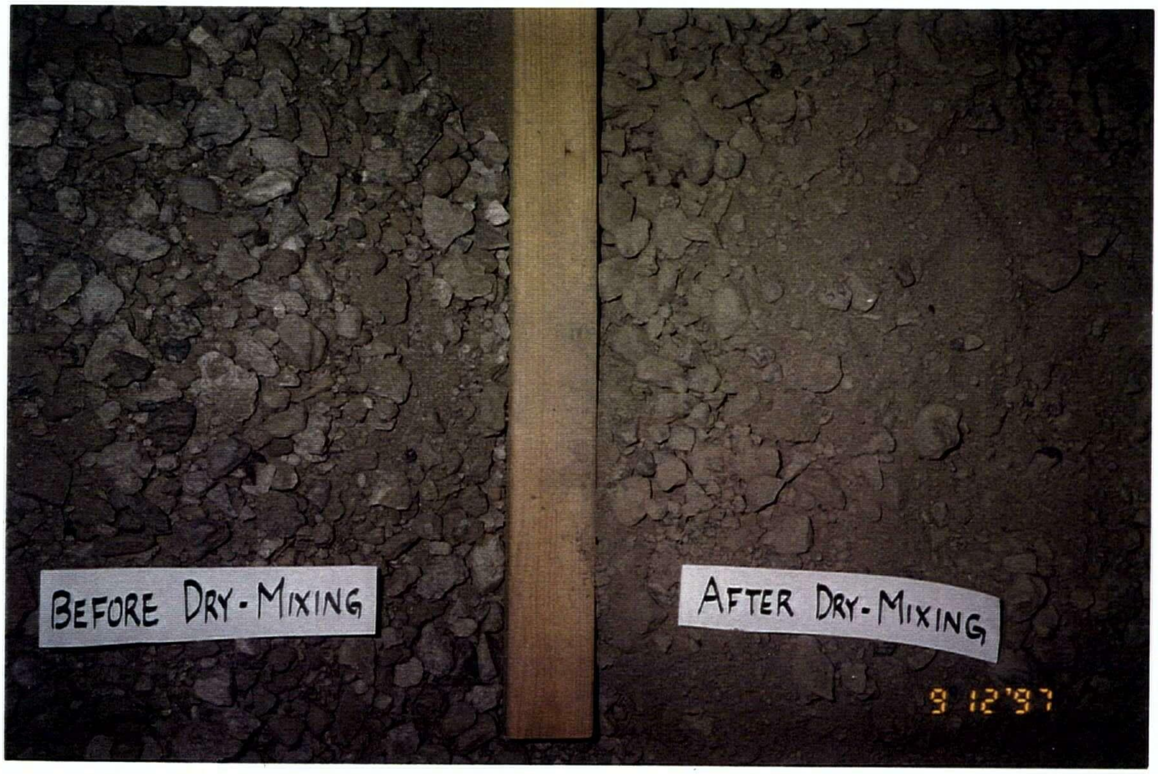


Figure 4.6 Comparison of aggregates before and after 25 min of dry-mixing.

other words, dry-mixing does not break the large particles but just removes the adhered material from their surfaces. This is because the original gravels are stronger than the mortar so all that breaks up during mixing is the mortar. In order to break large particles, the process would have to be changed by adding, for example, steel balls into the mixer which would further continue the breaking process.

The above descriptions of the changes that occur during the dry-mixing process can be graphically shown by gradation charts. As mentioned before, at every time of observation, a large enough sample of the material was obtained and graded. The particle size distribution at every time is shown in Table 4.4 and plotted in Figure 4.7. Note in Figure 4.7 that, with increasing dry-mixing time, the gradation curves gradually shift upward as a result of a finer gradation. The finer gradation, although not necessarily from the dust itself, can also be caused by the presence of broken up pieces from the larger aggregates. These pieces range in size from a fraction of a millimeter to a few millimeters. Finally, to further manifest the increasing fineness of the material with dry-mixing time, the fineness modulus, obtained from the gradation, is plotted against dry-mixing time, as shown in Figure 4.8. Although the changes in this parameter are not very large, a general downward trend is observed with increasing time in the mixer. This observation is in accordance with the observations made by Kasai [40]. In general, from all these results, it can also be concluded that, while the material fineness increases with increasing dry-mixing time, this increase is not very significant. There were no major changes in the gradation curves, or in the fineness modulus values, which again implies that dry-mixing may be beneficial in removing the attached mortar and in breaking up particles, but only to a limited extent.

Table 4.4 Gradation of recycled aggregates with dry-mixing time.

	Cumulative percent passing					
sieve number	at start*	after 3 min	after 6 min	after 10 min	after 15 min	after 25 min
3/4"	98	98	98	98	98	99
1/2"	81	79	80	80	79	83
3/8"	68	67	68	68	68	72
No.4	51	51	52	52	51	55
No.8	41	41	42	42	40	44
No.16	32	33	34	34	33	35
No.30	23	24	25	26	25	27
No.50	12	14	15	15	15	16
No.100	6	6	3	4	6	5
No.200	3	1	0	1	1	1

* This initial gradation is slightly different than the one given before because they were from another batch.

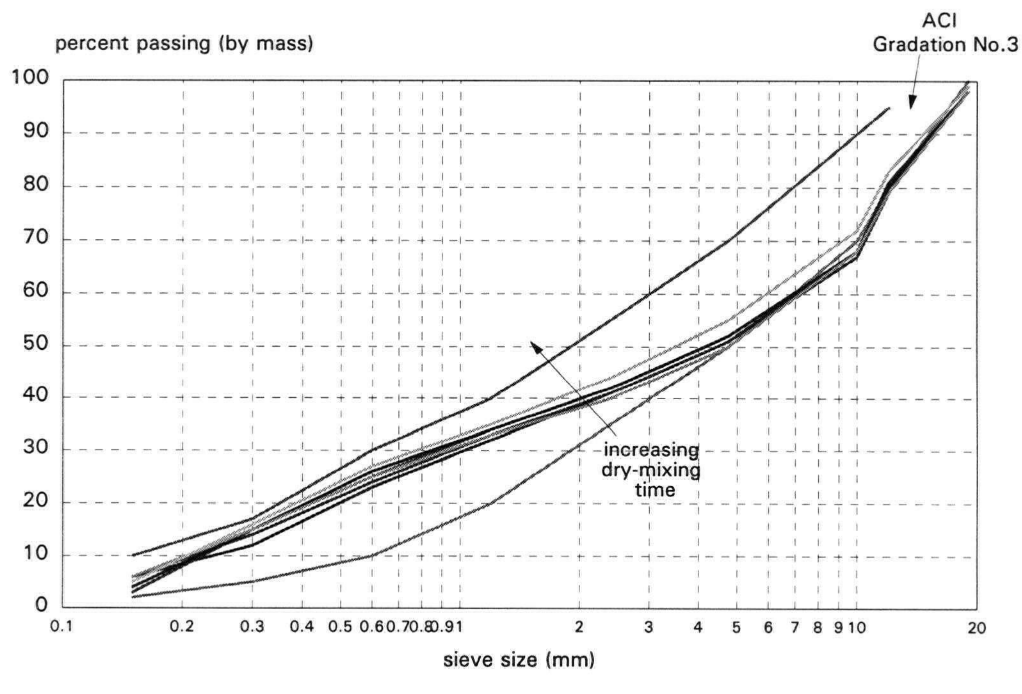


Figure 4.7 Changes in gradation of the recycled aggregates with increasing dry-mixing time.

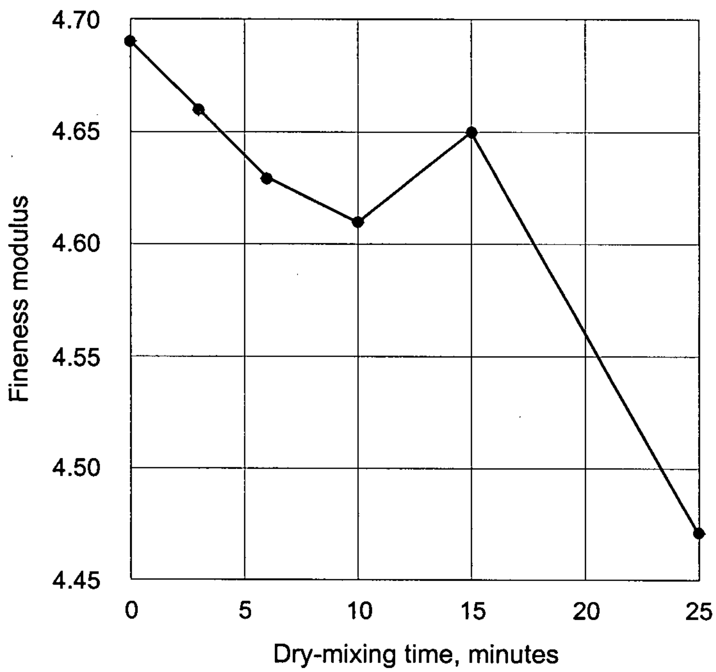


Figure 4.8 Changes in the fineness modulus of the recycled aggregates with increasing dry-mixing time.

5.0 EXPERIMENTAL WORK

5.1 Experimental Program

The objective of the research was to compare a number of performance variables between shotcrete made with recycled aggregates and shotcrete made with virgin aggregates. The two different shotcreting processes, dry-mix and wet-mix, were investigated. At the same time, since it is desirable to see if the same effects of the recycled aggregates occur in shotcrete as in cast concrete, companion concrete specimens were also cast. The experimental program is outlined in Figure 5.1. As seen, a pilot study was also carried out to examine the behaviour of fiber reinforced shotcrete made with recycled aggregates. Only the wet-process shotcrete was investigated with fiber in this pilot study.

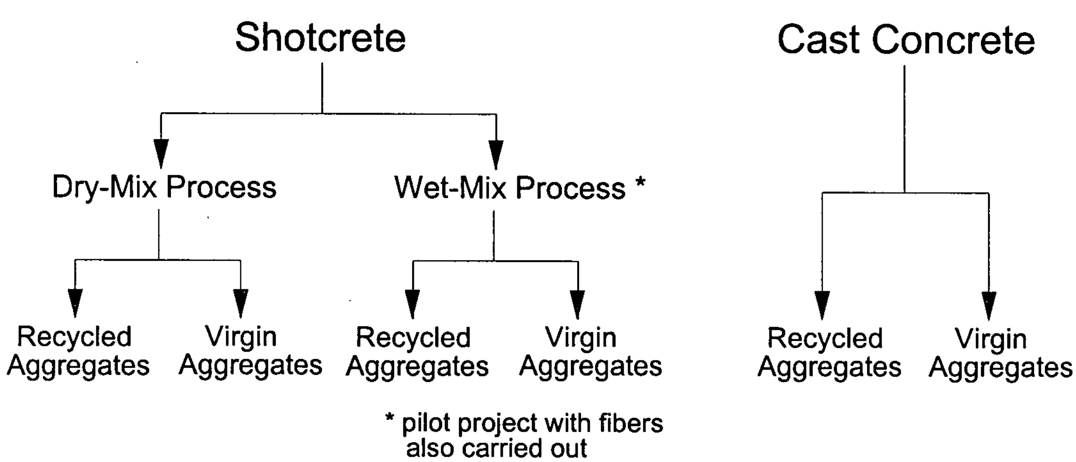


Figure 5.1 Experimental program.

5.2 Materials and Proportions

5.2.1 Materials

Cement

CSA Type 10 (ASTM Type I) normal Portland cement was used throughout the research.

Aggregates

Comparing the performance of shotcrete and concrete made with aggregates from different types of sources (recycled crushed waste concrete and naturally quarried) was the prime objective of the research. Since, at the time the experiments were performed, Columbia Bitulithic Ltd. was conducting tests on the crushers to produce different gradations of crushed material, it was decided to haul, at one time, a large enough pile of aggregates to prevent any discrepancies in gradation and composition among separate batches during the research study. The characteristics and properties of this crushed waste concrete have already been described in detail in Chapter 4. For purposes of mix proportioning in this investigation, it is worth to mentioning again that the nominal maximum size of the recycled aggregates is 3/4" (20 mm) and its overall (combined coarse and fine fractions) effective (30 min.) water absorption is 5.1%.

The virgin aggregates employed were actually a blend of different natural aggregate components. The purpose of this blending was to achieve a gradation of virgin aggregates which would simulate as closely as possible the gradation of the recycled aggregates in a given batch. In this way, the variability in results introduced by dissimilar gradations is eliminated from the process. Four separate components, all readily available from the lab, had to be blended in order to produce the desired results. These components were:

1) Lafarge Torpedo aggregate
2) Target normal CSA concrete sand

3) Target Forestry sand
4) Target Superfine sand.

The particle size gradation of these components is given in Table 5.9. The result was a blend with a nominal maximum aggregate size also of 3/4" and an overall (all components together) effective (30 min.) water absorption of 1.5%.

The method in which the recycled and virgin aggregates were processed and prepared before their utilization in the mixes is explained in detail in Section 5.3. Also, the resulting gradations are shown there.

Superplasticizer

The commercially available superplasticizer WRDA-19 was used in the wet mixes to aid in the plasticity of the mix. This superplasticizer is a solution belonging to the naphthalene sulphonate formaldehyde chemical family with a solid content of 40% and a specific gravity of 1.21.

Fibers

Fibers were used in a pilot project study to examine the flexural behaviour of macro-fiber reinforced shotcrete made using recycled aggregates. Only wet-mix shotcrete was investigated. The fiber used was a 30 mm hooked-end steel fiber with a circular cross-sectional shape. Other fiber characteristics included:

diameter	=	0.5 mm
tensile strength	=	1115 MPa
mass	=	45 mg (per fiber)
elastic modulus	=	210 GPa

5.2.2 Proportions

Since crushed waste concrete is highly variable in nature, there was no single pre-determined mix proportion that could be used for the experiments, particularly with respect to the water content. The two most important factors affecting the mix proportioning of recycled aggregate mixes are the higher water absorption which calls for higher water demands and the rougher, more angular texture of the aggregates which, in turn, also leads to harsher mixes. Thus, trial batches had to be performed with these aggregates in order to arrive at a mix deemed workable enough for shooting and for casting. Two of these trial batches were done, one for the dry-mix shotcrete and another for the wet mixes, either as wet-mix shotcrete or as cast concrete, as described below. In both cases, the cement and aggregate contents were set and water plus superplasticizer (in the case of the wet mixes) was added until a workable mix was obtained. Once the required water content was determined after accounting for the absorption of the recycled aggregates, mix proportions were derived for the virgin aggregate mixes taking into account the lower water absorption of the natural aggregates, such that the effective water-cement ratio between mixes with different aggregates was kept constant.

Dry-Mix Shotcrete

In dry-mix shotcrete, it is very hard to accurately predict the amount of water added. Judgement from the nozzleman is the key to control the volume of the water being forced through the water ring at the nozzle. This would be further expected to be different for the two types of aggregates. As for the dry materials, their relative proportions are shown in Table 5.1.

Table 5.1 Proportions of dry materials for dry-mix shotcrete.

Material	Amount (% of total dry materials, by mass)
cement	19%
aggregates (recycled or virgin)	81%

Wet-Mixes

The mix design was kept the same for both wet mixes, either as wet-mix shotcrete or as cast concrete. Since all the ingredients had to be mixed in a mixer prior to shooting or to casting, it was possible in the case of recycled aggregate mixes to add water gradually until a reasonable consistency was reached. This consistency was deemed to be plastic enough when the mix could be pumped through a hose and shot, or adequately consolidated by vibration on a table vibrator. Thus, trial batches were carried out for recycled aggregates and the resulting relative quantities for a workable mix, which were then used for the actual experiments are given in Table 5.2.

Table 5.2 Nominal proportions of materials for the wet mixes with recycled aggregates.

Material	Quantity
cement	440
aggregates (recycled)	1665
water	220
superplasticizer	10 ml/(kg of cement)

Based on the absorption of such aggregates compared to that of virgin aggregates, the relative mix quantities for the latter type were formulated and shown in Table 5.3.

Table 5.3 Nominal proportions of materials for the wet mixes with virgin aggregates.

Material	Quantity
cement	440
aggregates (virgin)	1665
water	160
superplasticizer	<10 ml/(kg of cement)

Thus, each of the above mix proportions were employed for the two types of wet mixes. In the fiber reinforced mixes, the proportions of materials were kept exactly the same as the ones shown in Tables 5.2 and 5.3, with the exception of an additional 78 kg/m^3 of steel fibers. This corresponds to a 1% by volume addition.

The above proportions are only nominal quantities which do not produce an exact cubic meter, given the difference in densities between the aggregate types and the different amount of water required in each case. The overyield or underyield factor can be determined by performing a solid volume calculation using the actual densities of the materials. These factors are then applied to the above quantities to obtain the true proportions in kg/m^3. As such, the true proportions for the wet-mixes are shown in Tables 5.4 and 5.5.

Table 5.4 Actual proportions of materials for the wet mixes with recycled aggregates.

Material	Quantity
cement	422 kg/m^3
aggregates (recycled)	1597 kg/m^3
water	211 kg/m^3
superplasticizer	10 ml/(kg of cement)

Table 5.5 Actual proportions of materials for the wet mixes with virgin aggregates.

Material	Quantity
cement	466 kg/m^3
aggregates (virgin)	1764 kg/m^3
water	169 kg/m^3
superplasticizer	<10 ml/(kg of cement)

As seen, the actual cement content for the virgin aggregate mixes is higher than that for recycled aggregate mixes. This higher cement content will tend to over-exaggerate the differences between virgin and recycled aggregate mixes for all test parameters evaluated.

5.3 Aggregate Preparation

5.3.1 Recycled Aggregates

Throughout the research, it was desired to minimize the extent of processing required for the recycled aggregates after these were obtained from Columbia Bitulithic Ltd. The reason for this is because any additional processing job would, in the industry, signify extra labour, time, and money. Therefore, the aggregates were used in a state as close as possible to the state in which they are found at the concrete recycling plant. The only aggregate processing performed on the recycled aggregates was the dry-mixing of these in a pan mixer for 25 min as this was considered to be an important factor in terms of enhancing the aggregate performance by stripping away the attached mortar, known to be a source of many problems. In the industry, this can easily be done by prearranging to dump the aggregates in a large mixer and run it for 20-30 min prior to mixing with the rest of the ingredients.

Thus, after the recycled aggregates were hauled from the recycling plant, only the following processes were carried out prior to batching:

1) Air-drying

   The aggregates had to be in, at least, an air-dried condition in order for them to be dry-mixed and graded. They were dried as needed prior to each batch.

2) Dry-mixing

   Once the aggregates were deemed to be dry enough, they were placed in a pan mixer and the mixer was allowed to run for 25 min. This process removed most of the surface adhered mortar and produced smoother particle shapes while increasing the dustiness of the mix. Again, the aggregates were dry-mixed as needed prior to each batch. It is important to mention here that, although different batches required different quantities of aggregates, the same amount was placed in the mixer to dry-mix every time aggregates were being prepared for a given batch. This is because the extent to which dry-mixing would have an effect on the characteristics of the aggregates would also depend on the size of the sample being dry-mixed. Any extra material was just discarded afterwards. Therefore, by dry-mixing the same amount of aggregates, this variable would be eliminated and consistency among the aggregates going to different batches could be maintained.

3) Grading

After dry-mixing the recycled aggregates, they were graded. This was done in order to enable one to know the exact particle size distribution going into each mix and to make sure that these complied with the gradation limits stipulated by standards. For the aggregates used in the dry-mix shotcrete, they also had to be screened off to eliminate sizes larger than 10 mm in order that its gradation fell within the ACI 506 gradation limits.

Other processes that could have been performed include: selective removal of contaminants; fine-sized particle screening; water washing to remove dust, fine material and light material; or air blowing to remove dust. These processes were, however, not done for the reasons mentioned before.

At the end of the above processes, the aggregates were ready to be used for either shooting or casting depending on the purpose for which they were prepared. The final dry-mixed and screened (in the case of dry-mix shotcrete aggregates) aggregate gradations for each of the different mixes are shown in Tables 5.6, 5.7 and 5.8. These gradations, in relation to their respective standards, are shown in Figures 5.2, 5.3 and 5.4. Note the closeness in gradation between aggregates used for wet-mix shotcrete and aggregates used for cast concrete, as these two materials were processed in the same manner.

Table 5.6 Gradation of aggregates used for dry-mix shotcrete.

sieve number	Cumulative percent passing
3/4"	100
1/2"	100
3/8"	93
No.4	70
No.8	59
No.16	48
No.30	37
No.50	20
No.100	11
No.200	2

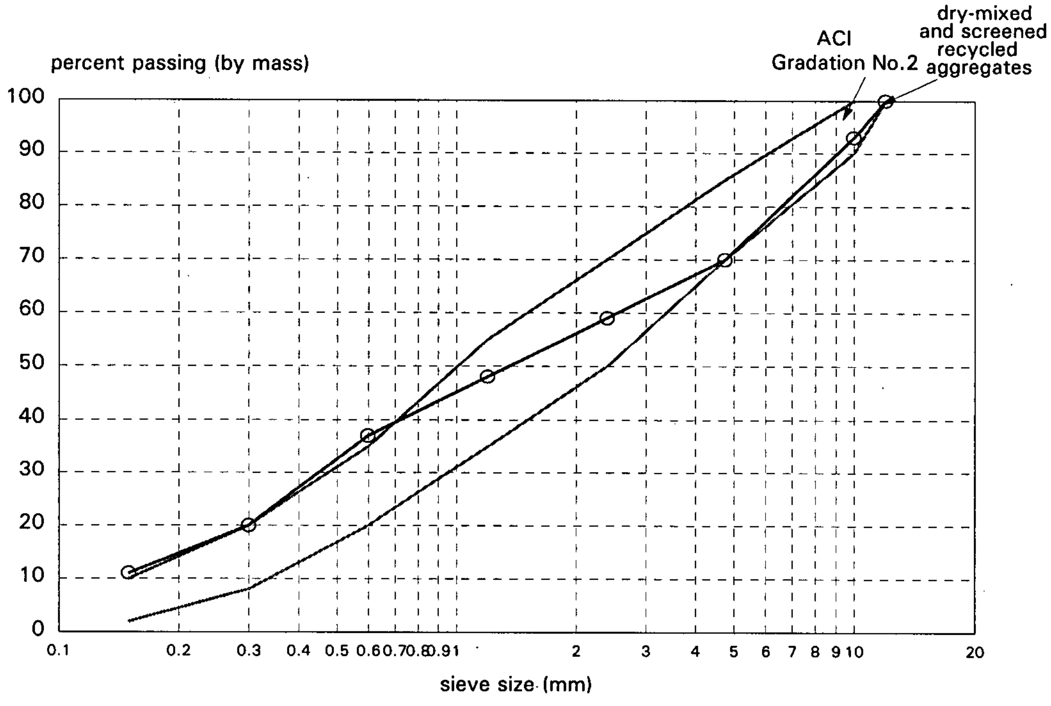


Figure 5.2 Gradation of aggregates used for dry-mix shotcrete.

Table 5.7 Gradation of aggregates used for wet-mix shotcrete.

sieve number	Cumulative percent passing
3/4"	99
1/2"	83
3/8"	72
No.4	55
No.8	44
No.16	35
No.30	27
No.50	16
No.100	5
No.200	0

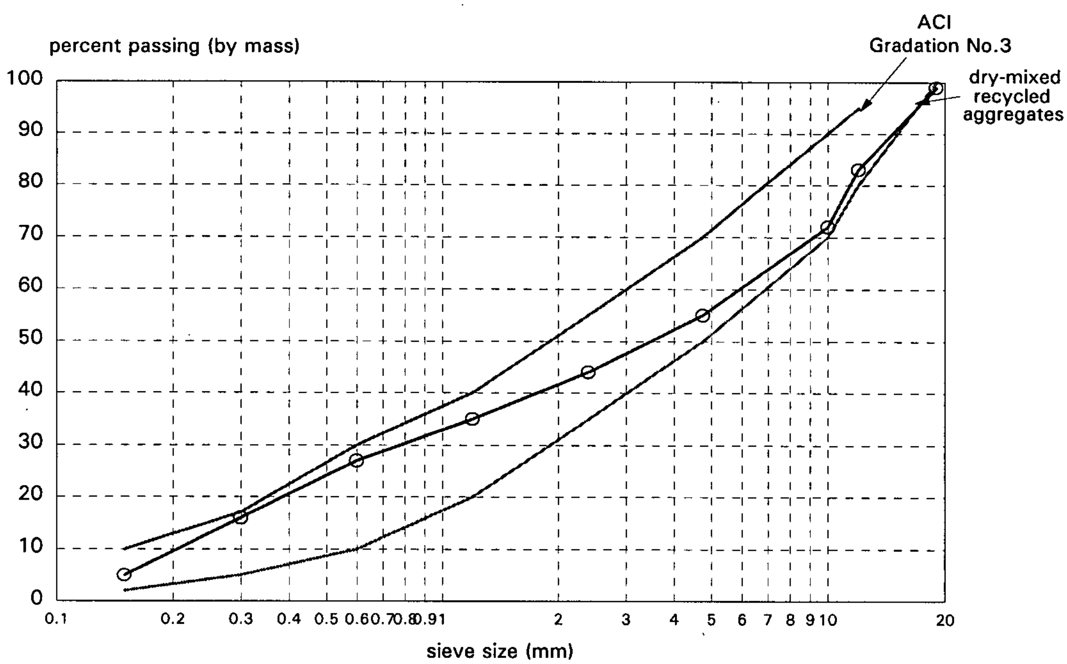


Figure 5.3 Gradation of aggregates used for wet-mix shotcrete.

Table 5.8 Gradation of aggregates used for cast concrete.

sieve size	Cumulative percent passing
20 mm	98
14 mm	84
10 mm	74
5 mm	56
2.5 mm	44
1.25 mm	35
630 μm	26
315 μm	16
160 μm	7
80 μm	2

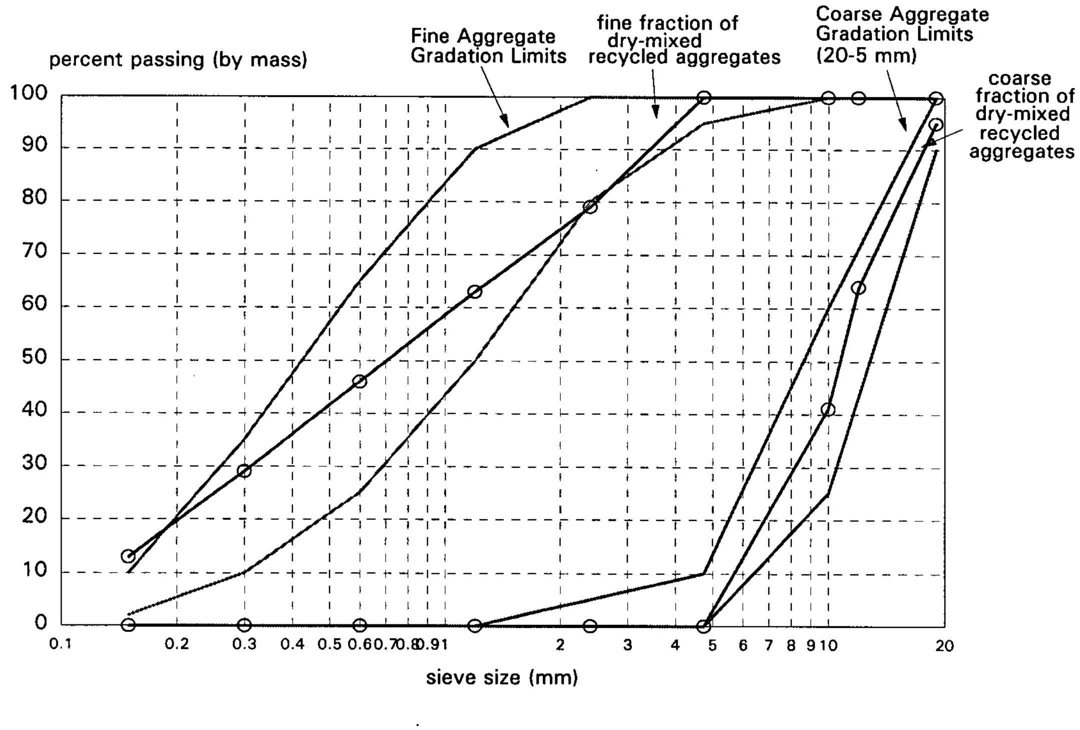


Figure 5.4 Gradation of aggregates used for cast concrete.

5.3.2 Virgin Aggregates (Regression Blending)

Most of the research that has been carried out to investigate the performance of recycled aggregate concrete to date does not indicate the specific gradation of the aggregates employed in the mixes. Aggregate gradation can, in fact, play a significant role in determining much of the resulting concrete properties. This could either be in an indirect manner by affecting the workability of the mix, which in turn causes differences in the internal concrete structure, or in a direct manner by affecting the matrix-aggregate bond. Even if concrete properties were not too seriously affected, the cost of the materials could become a determining factor; and, gradation limits are normally set to also take into account the economy of the mix. It could possibly be that the discrepancies observed between recycled and virgin aggregates mixes in the previous works were partly due to gradation differences. In this research, however, one of the key components was the preservation of a similar gradation between the two types of aggregates for a given mix. By doing so, the recycled and the virgin aggregates were comparable from a particle size distribution perspective and the main difference would then stem from their particular nature, which is the main target of this investigation.

Since minimal processing of the recycled aggregates was desired in order to have them in a state as close as possible to that found at the recycling plant, the virgin aggregates had to be manipulated in a way to produce aggregates of similar gradations. Another way of viewing the situation is to consider what would happen if the recycled aggregates that were to be used in a given application were replaced with natural aggregates of the same gradation. The problem then becomes one of attempting to make the gradation of the virgin aggregates conform as closely as possible to the gradation of the dry-mixed recycled aggregates. Another constraint to this matching procedure is the fact that it had to be done with material available from the lab because aggregates suppliers would not provide the required graded mix unless it was a large batch.

In order to obtain a virgin aggregate gradation comparable to that of the recycled aggregates, individual virgin components had to be mixed in certain proportions to achieve the desired resulting grade. Four components were required to produce a blend with a gradation similar to that of the dry-mixed recycled aggregates. These four components along with their significance are as follows:

1)	Lafarge Torpedo (3/4" maximum size):	This component was necessary to provide coarse particles in the upper range of the gradation curve of recycled aggregates. It was particularly required for the wet-mixes whose gradation curves extended to these sizes.
2)	Target normal CSA concrete sand:	This component was necessary to provide the continuous gradation of sizes ranging from ones slightly over the No.4 sieve to the fine range. Without this component, the resulting blend was gap-graded at several locations in the gradation curve.
3)	Target Forestry sand:	This component was required to maintain a balance between the coarser end and the finer end of the gradation. Given the fact that there is a significant size difference between the coarse end of the scale and the fine aggregates, the intermediate sizes would be missing. This Forestry sand, being a mix of uniform size in that intermediate range was the most suitable component to add to the blend.

4) Target Superfine sand: This component was required to simulate the fine material contained in the recycled aggregates after they were dry-mixed. It did not affect the coarser ranges very much but did elevate the fine fractions to substantial levels.

All these individual components were available in the lab at the time of the processing. Their gradations had to be determined prior to blending, and they are shown in Table 5.9.

Table 5.9 Gradations of the individual virgin aggregate components.

	Cumulative percent passing			
sieve number	Lafarge Torpedo	Target CSA concrete sand	Target Forestry sand	Target Superfine sand
3/4"	100	100	100	100
1/2"	83	100	100	100
3/8"	28	100	100	100
No.4	0	100	100	100
No.8	0	94	68	100
No.16	0	75	1	100
No.30	0	40	0	100
No.50	0	13	0	34
No.100	0	4	0	4
No.200	0	0	0	1
FM	6.7	2.7	4.3	1.6

Once the gradations of these individual components were obtained, the next task was to find the proportions of these that would produce the desired gradation.

A method commonly employed for the blending of aggregates is the "graphical method" [62]. In this method, the gradation of the components are plotted in a chart along with the required limits for the resulting blend, and by drawing lines in a specified manner on the chart, the range of individual proportions for the components can be determined. This method, however, was found unsuitable for the purposes of blending the virgin aggregate components in these experiments. The main reasons for this inadequacy are:

1) For the blending of more than two components, the space of the "chart" in which the values are plotted would have to be higher than two dimensions, making it extremely difficult to represent;
2) For the experiments, it is desired to obtain a gradation of blend as close as possible to an existing one. This would imply that the required limits on the chart must be within very narrow ranges to satisfy that existing gradation. When the blend cannot suit this narrow range, this method contained a number of singularities.

Thus, this aggregate blending approach was discarded.

The alternative method used to arrive at the required proportions for blending was based on a regression style analysis of minimizing the total error between the theoretical grading produced by blending the individual components in certain proportions and the desired grading. The desired grading here is, as mentioned before, the grading of the dry-mixed recycled aggregates. The theoretical grading of the blend is one that can be obtained by the following expression:

for a given sieve number x, $$P_x = \sum_{i=1}^{n} p_{xi} * w_i \qquad (5.1)$$

where P_x is the cumulative percent passing that sieve number x in the total blend;
p_{xi} is the cumulative percent passing that sieve number x in component i;
w_i is the weight fraction of component i in the total blend;
n equals the total number of components blended together;

x	designates a particular sieve size (e.g. No.4, 3/8", etc.) and ranges over the number of sieves used.

The summation in the equation is carried out for all components from the first one to the n^{th} one to obtain P_x. Thus, by knowing the gradation of the individual components and the proportions at which these components appear in the blend, Equation 5.1 can be applied in turn to each sieve size to construct the complete gradation of the final blend. It should also be noted that:

$$\sum_{i=1}^{n} w_i = 1$$

In the experiments, prior to the actual blending, the values of p_i are known from the gradation of the individual components, as shown in Table 5.9, but those of w_i are unknown. So, in the analysis, the minimization of the error between the two gradations was used as the criterion to arrive at the values of w_i.

If the desired target gradation is labeled D, the cumulative percent passing a given sieve number x in this gradation would then be D_x. The error in gradation between the theoretical blend and the target blend at a particular sieve size, x, can therefore be given by:

$$\varepsilon_x = P_x - D_x \tag{5.2}$$

This is graphically represented in Figure 5.5.

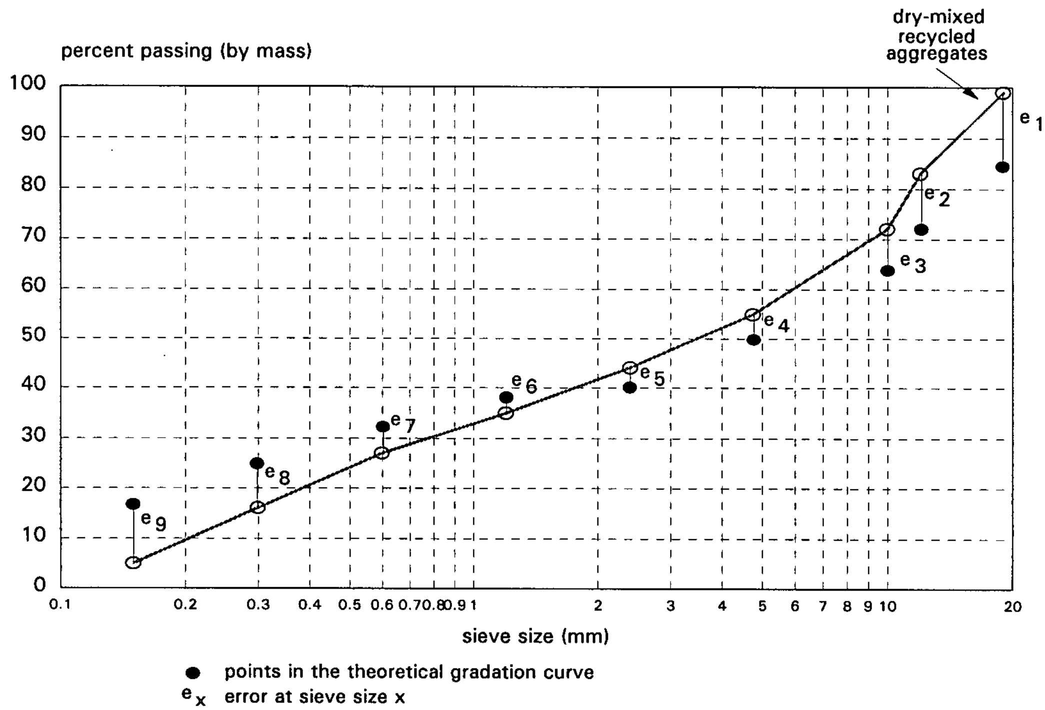


Figure 5.5 Graphical representation of the meaning of the errors.

The objective of the analysis is to minimize the sum of all the errors over the range of the sieve sizes covered. A more convenient approach would be to minimize the sum of the errors squared. Thus, the objective function can be written as follows:

$$\text{minimize} \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} \varepsilon^2 = \text{minimize} \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} (P_x - D_x)^2 = \text{minimize} \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} \left(\sum_{i=1}^{n} p_{xi} * w_i - D_x\right)^2 \qquad (5.3)$$

Note that, at this point, the number of components n has still been left general and the blend can include as many components as possible.

To minimize the sum of the errors squared as shown by Equation 5.3, the partial derivatives of the expression have to be taken with respect to each of the variables which, in this case, are w_i. However, before doing so, another expression can be introduced into the equation. This expression sets the constraint that all the mass fractions must add up to unity and is given by:

$$\sum_{i=1}^{n} w_i = w_1 + w_2 + \ldots + w_{n-1} + w_n = 1 \tag{5.4}$$

So, any one w_i can be selected and expressed in terms of the others. This equation is then inserted back into Equation 5.3, forcing that condition and also reducing the number of variables by one.

Thus, the overall steps can be summarized as follows:

1) write out the objective function (Equation 5.3) for all components n to be covered;
2) select any w_i and express it in terms of the others w_j, $i \neq j$, in order to satisfy their unity sum condition;
3) insert this into the objective function;
4) set the partial derivatives of this function with respect to the remaining variables to zero;
5) solve for these variables and then go back to solve for the isolated variable in 2);
6) using the now known values of wi and the known component gradations, p_i, use Equation 5.1 to calculate the theoretical grading with minimal error.

The above process was generalized for two, three and four components. Since the experiments covered four virgin components as listed before, only the generalized expressions for the four component blend are presented here.

Let C_x be the cumulative percent passing sieve size x in the coarse (C) aggregate (Lafarge Torpedo gravel) gradation,
F_x be the cumulative percent passing sieve size x in the fine (F) aggregate (Target CSA concrete sand) gradation,
U_x be the cumulative percent passing sieve size x in the uniform (U) size aggregate (Target Forestry sand) gradation,
SF_x be the cumulative percent passing sieve size x in the superfine (SF)

aggregate (Target Superfine sand) gradation,

D_x be the cumulative percent passing sieve size x in the desired (D) aggregate gradation.

and

w_c be the mass fraction of the coarse (C) aggregate in the final blend,

w_f be the mass fraction of the fine (F) aggregate in the final blend,

w_u be the mass fraction of the uniform size (U) aggregate in the final blend,

w_{sf} be the mass fraction of the superfine (SF) aggregate in the final blend.

Then, the optimum values of w_c, w_f, w_{sf}, and w_u, such that the error is minimum, are given by:

$$w_f = \frac{(BG - DH)A\alpha + A\beta\theta + G\beta^2}{A(AB - D^2)\alpha}$$

$$w_c = \frac{(ADH - D^2G)\alpha + G\beta\gamma + A\gamma\theta}{D(AB - D^2)\alpha} \tag{5.5}$$

$$w_u = \frac{-G\beta - A\theta}{\alpha}$$

$$w_{sf} = 1 - w_f - w_c - w_u$$

where

$$\alpha = (AB - D^2)(AC - E^2) - (AF - DE)^2$$
$$\beta = E(AB - D^2) - D(AF - DE)$$
$$\gamma = D(AF - DE)$$
$$\theta = HAF - HDE - ABI + ID^2$$

where

$$A = \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} (F_x - SF_x)^2$$

$$B = \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} (C_x - SF_x)^2$$

$$C = \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} (U_x - SF_x)^2$$

$$D = \sum_{\substack{\text{all sieve}\\ \text{sizes x}}} (F_x - SF_x)(C_x - SF_x)$$

$$E = \sum_{\substack{\text{all sieve} \\ \text{sizes } x}} (F_x - SF_x)(U_x - SF_x)$$

$$F = \sum_{\substack{\text{all sieve} \\ \text{sizes } x}} (C_x - SF_x)(U_x - SF_x)$$

$$G = \sum_{\substack{\text{all sieve} \\ \text{sizes } x}} (F_x - SF_x)(D_x - SF_x)$$

$$H = \sum_{\substack{\text{all sieve} \\ \text{sizes } x}} (C_x - SF_x)(D_x - SF_x)$$

$$I = \sum_{\substack{\text{all sieve} \\ \text{sizes } x}} (U_x - SF_x)(D_x - SF_x)$$

With the above expressions in a readily computable form and knowing the gradations of the four virgin components, as well as the desired target gradation for each batch, the required proportions of the component aggregates were determined. Having these values, it was possible to calculate what the theoretical gradations for each of the mixes were. Simply calculating these theoretical gradings, however, is not sufficient, as it is important to check the validity of the model by performing actual gradation analysis on the blended material in order to: (a) compare it with the theoretical gradations, and (b) compare it with the desired gradations, which is the purpose of the whole exercise. These gradations are shown for the dry-mix shotcrete aggregates in Table 5.10, for the wet-mix shotcrete aggregates in Table 5.11, and for the cast concrete aggregates in Table 5.12. The actual gradations are also shown graphically along with that of the desired target gradations for the dry-mix shotcrete aggregates in Figure 5.6, for the wet-mix shotcrete aggregates in Figure 5.7, and for the cast concrete aggregates in Figure 5.8.

Table 5.10 Comparison of the dry-mix shotcrete aggregate gradations.

	Cumulative percent passing		
	Virgin Aggregates		Recycled Aggregates
sieve number	Theoretical Blend (from analysis)	Actual Gradation (experimental)	Desired Gradation (from Table 5.6)
3/4"	100	100	100
1/2"	100	100	100
3/8"	85	86	93
No.4	71	71	70
No.8	64	64	59
No.16	47	46	48
No.30	40	40	37
No.50	14	14	20
No.100	2	2	11
No.200	0	0	2

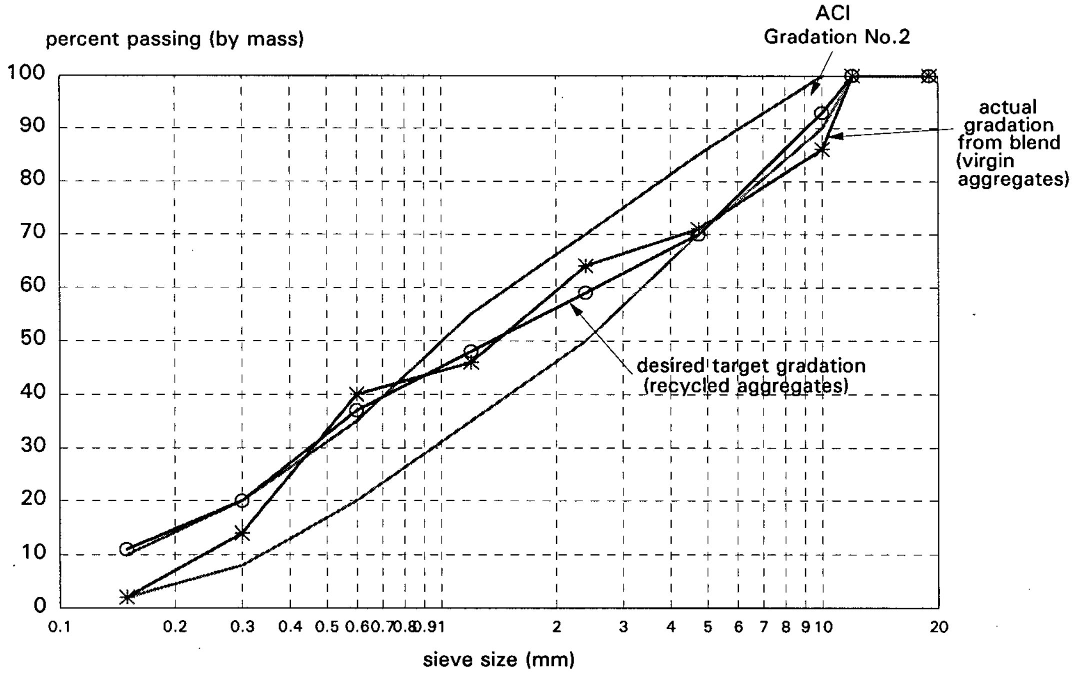


Figure 5.6 Comparison of the recycled and virgin aggregate gradations used for dry-mix shotcrete.

Table 5.11 Comparison of the wet-mix shotcrete aggregate gradations.

sieve number	Cumulative percent passing		
	Virgin Aggregates		Recycled Aggregates
	Theoretical Blend (from analysis)	Actual Gradation (experimental)	Desired Gradation (from Table 5.7)
3/4"	100	100	99
1/2"	92	90	83
3/8"	67	64	72
No.4	54	51	55
No.8	48	46	44
No.16	34	30	35
No.30	29	27	27
No.50	10	9	16
No.100	1	1	5
No.200	0	0	0

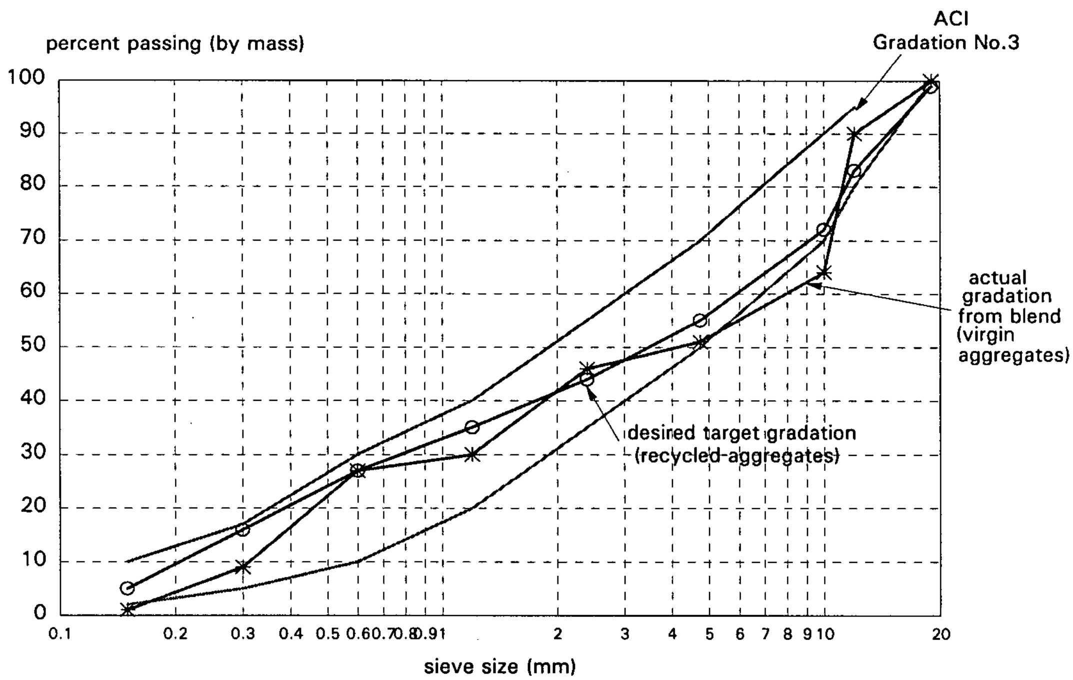
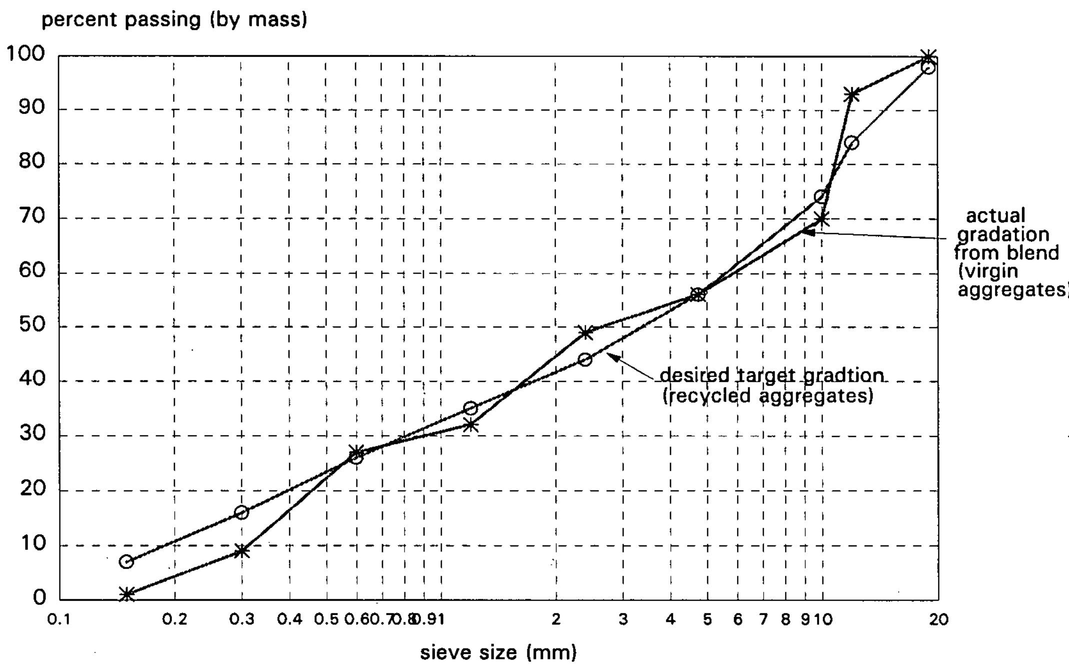


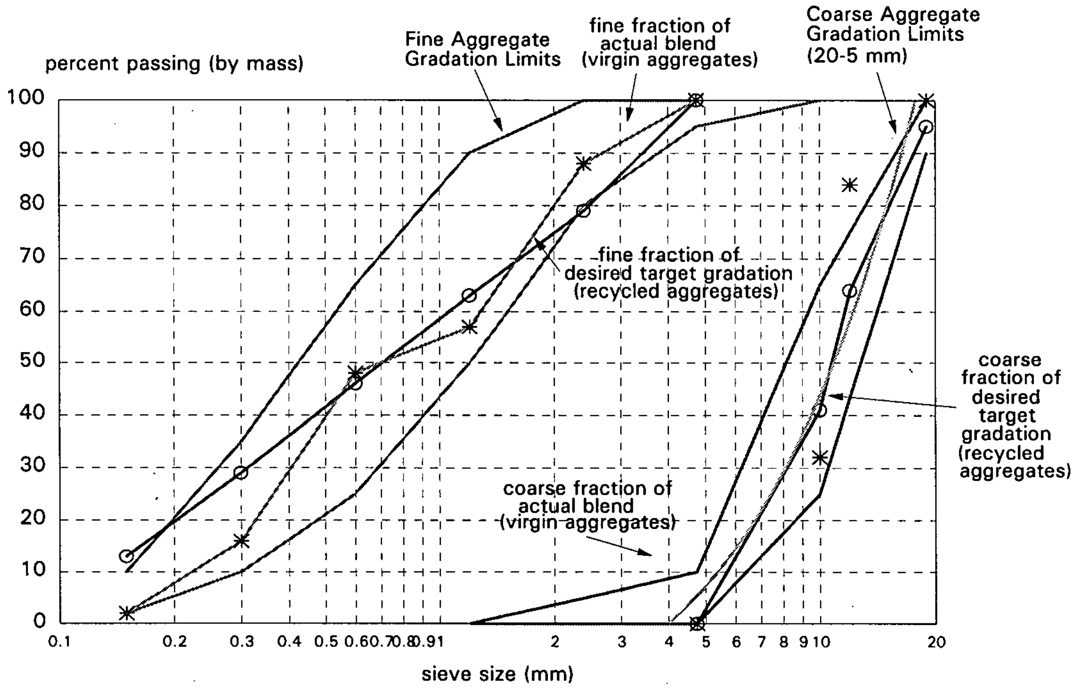
Figure 5.7 Comparison of the recycled and virgin aggregate gradations used for wet-mix shotcrete.

Table 5.12 Comparison of the cast concrete aggregate gradations.

	Cumulative percent passing		
	Virgin Aggregates		Recycled Aggregates
sieve size	Theoretical Blend (from analysis)	Actual Gradation (experimental)	Desired Gradation (from Table 5.8)
20 mm	100	100	98
14 mm	92	93	84
10 mm	68	70	74
5 mm	56	56	56
2.5 mm	49	49	44
1.25 mm	34	32	35
630 μm	28	27	26
315 μm	9	9	16
160 μm	1	1	7
80 μm	0	0	2



(a)



(b)

Figure 5.8 Comparison of the recycled and virgin aggregate gradations used for cast concrete: (a) complete gradation curve and (b) in relation to CSA A23.1.

From the above, it can be seen that, in general, the resulting actual gradations of the blends are very similar to the analytical ones, thus proving the validity of the method of analysis. Furthermore, these resulting gradations are quite close and comparable to the desired target ones. Some deviations occur at certain points but the overall trend and range of values are within agreement. The reasons for these deviations are mainly due to sampling problems. However, they may not turn out to be critical at all, particularly in the case of shotcrete where material rebounding invariably changes the in-situ aggregate gradation. Nevertheless, by going through the exercise and process described in this section, at least, the shot material has comparable properties from a particle size distribution standpoint for both recycled and virgin aggregates.

It should also be mentioned that the error minimization procedure was also attempted for different size ranges of the gradation curve. The idea behind this is that, in shotcrete, most of the coarse particles bounce off so there is not too much benefit in spending too much effort to get the coarse portion close to the desired curve. Therefore, the procedure outlined before was carried out only for the range of aggregates below 3/8" (10 mm). It turned out that the overall error was even larger in this case, most likely due because of the large deviations in the coarse range which also affected the fine range. This method was not adopted.

5.4 Shotcreting and Casting

5.4.1 Equipment

In the research, different processes were used to produce shotcrete or concrete made with either recycled or virgin aggregates (Figure 5.1). The following provides a description of the equipment used for each of the processes.

Dry-Mix Shotcrete

Dry-mix shotcrete for both aggregate types was produced using a dry-mix rotating barrel equipment (model ALIVA 246 with a 3.6 liter, eight pocket drum) instrumented with a spring-loaded, in-line, air flowmeter (model OMEGA FL8945). Water was added about 2 m before the nozzle at high pressure (between 1 and 5.2 MPa, read on a manometer and controlled by the nozzleman). Such high water pressure is needed to enable a better dispersion of water in the mix as well as a reduction in the variation in the water feed caused by fluctuations in the air pressure. In addition, a 20 m long hose, with an internal diameter of 50 mm, and maximum rotor speed were used in order to minimize fluctuations in the feed of material to the nozzle. An air flow of 300 cfm (100 cfm = 0.05 m^3/s) was used and the water pressure was adjusted as needed at the nozzle. Figure 5.9 shows a picture of this equipment.

Figure 5.9 Dry-mix shotcreting.

Wet-Mix Shotcrete

Wet-mix shotcrete for both aggregate types was produced using a wet-mix rotating barrel equipment (model ALIVA 262 with a 10 liter, twelve pocket drum). This machine corresponds to the pneumatic type of delivery. Here, the material travels through the hopper into the rotor chambers. The rotating drum then transports the material to the blasting chamber where compressed air forces the material through the hose and to the nozzle. A 5 m long hose, with an internal diameter of 50 mm, was used. The air flow was kept at 350 cfm and the rotor speed at a discharge of 4.7 m^3/hr. Figure 5.10 shows a picture of this equipment.

Figure 5.10 Wet-mix shotcreting.

<u>Cast Concrete</u>

The cast concrete specimens were made in an ordinary lab type rotating pan mixer. This is shown in Figure 5.11.

Figure 5.11 Cast concrete equipment.

5.4.2 Methods

This section describes the methods employed during specimen preparation in the different processes.

Dry-Mix Shotcrete

Dry-mix shotcrete with recycled or virgin aggregates was sprayed onto wooden forms (350 mm x 440 mm x 100 mm) with tapered sides inside a closed chamber (2.4 m x 2.4 m x 2.4 m). Water pressure was adjusted at the nozzle as necessary to produce a mix with well-wetted material. For both recycled and virgin aggregates, this pressure turned out to be very similar at around 450-460 psi (3.1-3.2 MPa). The nozzle was gyrated so that the stream moved in loops across the forms and the panel was shot from bottom to top. The shooting angle was kept as close to 90° from the vertical as possible in order to reduce rebound. The shooting distance was about 1 m.

Before each shoot, plastic tarpaulins were laid out on the chamber's floor to collect the rebound. This was weighed after every shoot, as well as the panel with material, to allow the calculation of rebound. Also, immediately after shooting, the shooting consistency was assessed using the electronic penetrometer developed at UBC [68]. This was done to ensure that the mixes had appropriate penetration stress and also to ensure that the recycled and virgin aggregate mixes had similar consistencies, in order for them to be on a comparable.

From each panel, a sample of fresh shotcrete was collected and heat dried after shooting to determine the in-situ water content of the mix. Also, large enough samples of the in-situ and the rebound materials were gathered and washed over a 75 μm sieve to obtain the in-situ cement content and the aggregate gradation of the in-situ and the rebound materials.

Wet-Mix Shotcrete

Similar to the dry-mix shotcrete methods, wet-mix shotcrete with recycled or virgin aggregates was sprayed onto the same wooden forms in the same chamber. The rotor speed was set to produce a discharge of 4.7 m^3/hr. The nozzle maneouvering techniques and shooting position were the same as for the dry-mix shotcreting.

Also, before each shoot, plastic tarpaulins were laid out on the chamber's floor to collect the rebound. This rebound was weighed after every shoot, as well as the panel with material, to allow the calculation of rebound. After mixing the material and before shooting, the slump of the mix and the air content were determined. Also, after shooting, large enough samples of the in-situ and the rebound materials were gathered and washed over a 75 μm sieve to determine the in-situ and rebound aggregate gradations.

In the mixes with fibers, the same procedures as described were carried out with the addition of fiber collection from the rebound material. This enabled calculation of the fiber rebound and thus, the in-situ fiber content.

Cast Concrete

Concrete mixes with recycled or virgin aggregates were also cast in the same wooden forms that were used for the shotcrete mixes. The reason for this was to maintain uniformity in the mixes produced by casting and by shooting, by eliminating any possible "wall effects" that cylindrical moulds would induce.

The aggregates followed by the cement, the water and the superplasticizer were added into the pan mixer. After each mix, the slump and air content were determined. The mix was then poured into the wooden forms and vibrated on a table vibrator.

After preparing the test panels in the methods described above, they were demoulded after 24 hrs and moist cured afterwards. At 3, 7, 28, and 96 days, the panels were cored to obtain 6 specimens for testing at those ages (3 for compressive stress-strain tests and 3 for splitting tensile tests). Additional samples were also taken at 28 days for the determination of ASTM C642boiled absorption and permeable voids content. The panels sprayed with fibers were sawn at 28 days to obtain 3 beams from each panel. Only 28 day tests were done for these specimens. Cores were 57 mm in diameter and 96 mm in height; while beams were 350 mm long with a 100 mm x 100 mm cross-section. These comply with the ASTM C1018-94b standards for specimen size for flexural toughness testing.

5.4.3 Data and Observations

As the specimens were prepared using the methods described in Section 5.4.2, relevant data was obtained. These data are all summarized in Table 5.13.

Note that the amount of rebound material in the wet-mix shotcrete is lower than in the dry-mix shotcrete, for both aggregate types. This is not surprising because dry-mix shotcrete almost always leads to higher rebound than its wet-mix counterpart.

It can be seen that the penetration stress of the recycled aggregate dry-mix shotcrete is almost the same as that of the virgin aggregate shotcrete. Because the water content is difficult to accurately control in dry-mix shotcrete, this value is used to assess the consistency between mixes. The two mixes are therefore comparable in this respect and variations due to water content should not be substantial. The fact that the water pressure at the nozzle was capable of being maintained at about the same level for both types of aggregates may indicate that, in the dry-mix process, there is not enough time for the recycled aggregates to soak up the water as they are being shot thus, resulting in an in-situ mix with similar consistency to that of virgin aggregate shotcrete.

The gradations of the aggregates in the in-situ and in the rebound material were determined for all the shootings. These were obtained from the material retained after

Table 5.13 Data obtained during the specimen making.

	Dry-mix shotcrete		Wet-mix shotcrete		Cast concrete	
	Recycled	Virgin	Recycled	Virgin	Recycled	Virgin
Total material rebound (%)	21	25	15	18	-	-
Fiber rebound (%)	-	-	25	32	-	-
In-situ fiber content (vol %)	-	-	0.91	0.89	-	-
In-situ cement content (%)	not determined	20	-	-	-	-
In-situ water-cement ratio	not determined	0.42	-	-	-	-
Effective water-cement ratio	-	-	0.32	0.32	0.32	0.32
Penetration resistance (MPa)	2.5	2.5	-	-	-	-
Slump (mm)	-	-	52	52	27	27
Air content (%)	-	-	not determined	not determined	1.6	1.7
Boiled absorption (%) [1]	12.7 (marginal)	7.5 (good)	11.7 (marginal)	7.0 (good)	11.3 (marginal)	6.9 (good)
Permeable voids content (%) [1]	25.1 (marginal)	16.7 (good)	24.0 (marginal)	16.1 (good)	23.9 (marginal)	16.0 (good)

[1] descriptor in parenthesis represents the quality of the material with such values of boiled absorption and permeable voids content.

washing representative samples over the 75 μm sieve. These gradations are approximate because there is a significant amount of fine material smaller than 75 μm that also gets washed away, but for comparative purposes, the results are sufficiently accurate over the entire range of sizes. These gradations are shown in Figure 5.12 for dry-mix shotcrete and in Figure 5.13 for wet-mix shotcrete. The recycled and virgin aggregate mixes for each process are shown in the same graphs for comparison. It is seen that, in either shotcrete process, the in-situ gradations for recycled aggregates and for virgin aggregates are fairly close, indicating that the in-situ materials do in fact have comparable particle size distributions, and variations due to this should only be minor.

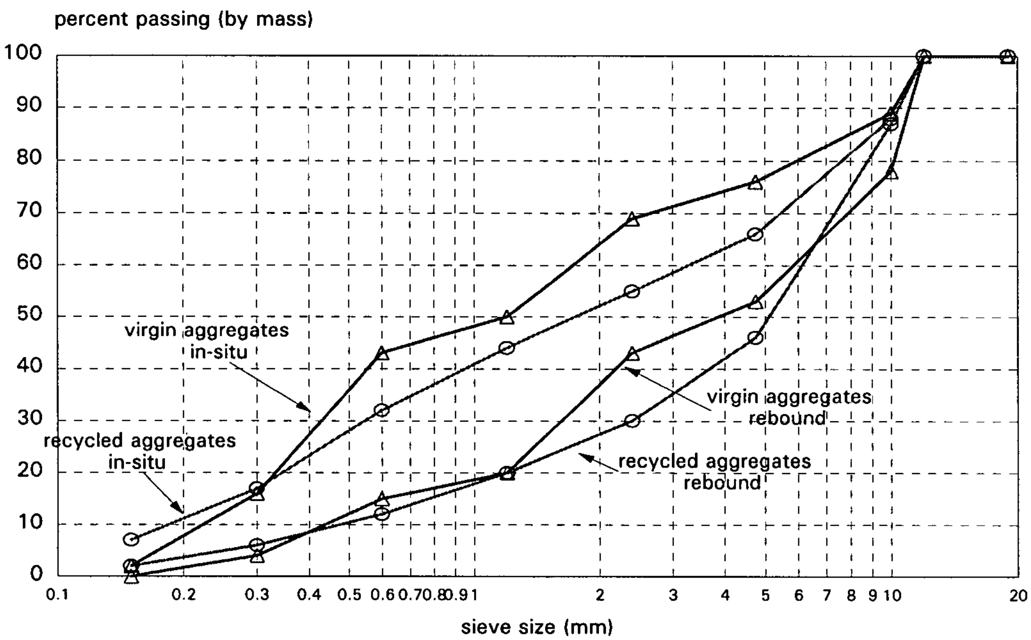


Figure 5.12 Gradation of aggregates in in-situ and in rebound material for dry-mix shotcrete.

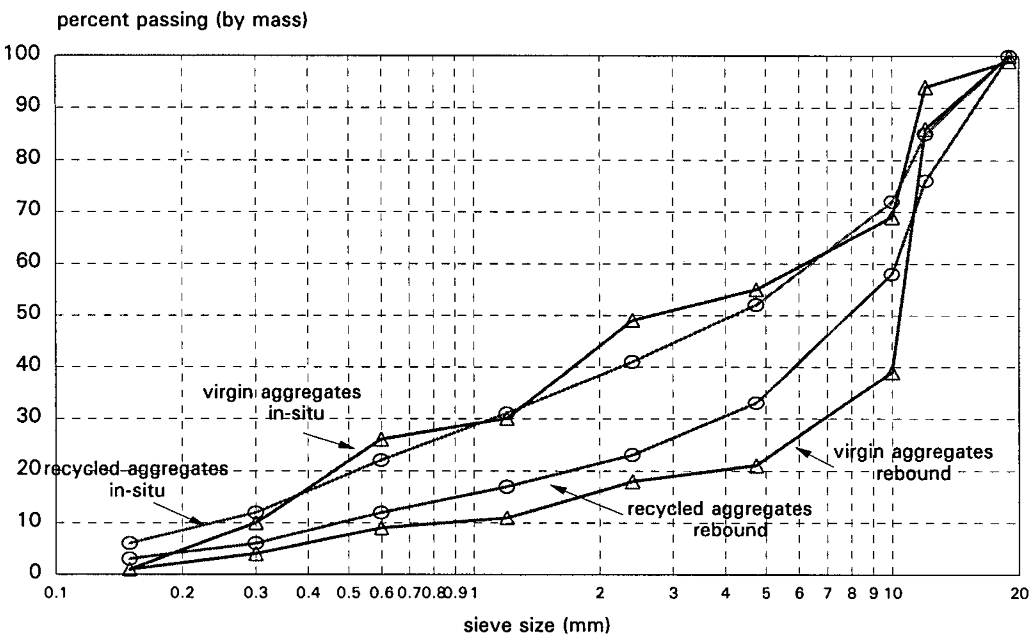


Figure 5.13 Gradation of aggregates in in-situ and in rebound material for wet-mix shotcrete.

One very significant observation made during shooting was that recycled aggregate mixes tended to be very cohesive and "gummy" resulting in a better buildup of shotcrete with less rebound. This is most likely due to the higher water absorption of the aggregates in addition to the high surface area created by the large levels of dust in the mix. Although the extra water needed to saturate the aggregates was already taken into account in the mix design, during the time that this happens, the mix stiffens up. The other factor is the high surface area which causes the adsorption of water onto the surface of the fine material. This adsorbed water is then no longer available to provide lubrication to the mix as it is attached somewhere else. As a result, the mix experiences a rapid loss of workability while maintaining a cohesive nature. This property proves to be advantageous in the case of shotcrete for both the dry-mix and wet-mix processes, because higher buildups with less rebound can be achieved, as observed during shooting (see Table 5.13). It not only applies to normal shotcrete but also to fiber reinforced shotcrete where a lower fiber rebound was also observed in recycled aggregate shotcrete leading to a higher in-situ fiber content (see Table 5.13). In a way, the benefits imparted by the fine material on the plastic properties of shotcrete are somewhat comparable to those provided by silica fume, i.e. a higher cohesiveness with less rebound.

A slight problem experienced during the shooting of recycled aggregate mixes was an intermittent hose blocking. This was probably a result of the angular nature of the aggregates causing them to get stuck at times as they traveled through the hose; and, as a consequence, the flow of material came as pulses. If severe, it could lead to serious problems in dry-mix shotcrete because, even if the flow of material is slowed down, water is continually sprayed and could possibly lead to the formation of water pockets in the shotcrete. This problem could be corrected by using larger hoses, if available, or by using smaller maximum size aggregates, or by using superplasticizers, in the wet-mix case, to provide more lubrication into the mix. If superplasticizers are used, care must be taken in controlling the quantity added because excessive amounts of these can cause the shot mix to slough off the substrate.

The boiled absorption and permeable void content values of recycled aggregate shotcrete and concrete are higher in all cases than those of virgin aggregate mixes. At these levels, the recycled aggregate mixes can only be categorized as poor in terms of their quality. The higher values are indicative of a larger porosity in the internal structure which is most likely caused by the higher porosity of the old matrix material and the contaminants. The large dust content and high initial water-cement ratio may have also resulted in inadequate mixing at locations distributed through the matrix which, on the hydration of the paste, became void spaces. However, such marginal quality material is not of particular concern in temporary applications where only minimal quality is required from the material. In such cases, recycled aggregate shotcrete or concrete.

5.5 Testing Program

5.5.1 Stress-Strain Tests

Stress-strain tests were performed on cores 57 mm diameter and 96 mm high obtained from the panels. This specimen geometry meets the requirements of ASTM C469 for modulus of elasticity tests. The tests were done on a 150 kN floor-mounted Instron materials test system with the load being recorded by the load cell on the machine and the displacement being recorded by two LVDT's mounted on opposite sides of the specimen. The gauge length for displacement measurements was set at two-thirds of the specimen height which is 64 mm. The signals from the load cell and the LVDT's were sent to a data acquisition system and recorded at a rate of 1 Hz. This setup is shown in Figure 5.14 and schematically in Figure 5.15.

Trial tests which were done prior to the actual ones showed that stability of the tests leading to the acquisition of a complete stress-strain response (with softening branch) was possible at very slow loading rates. This rate was set to a cross-head displacement rate of 0.01 mm/min. However, the modulus of elasticity value in accordance to the ASTM C469 standard

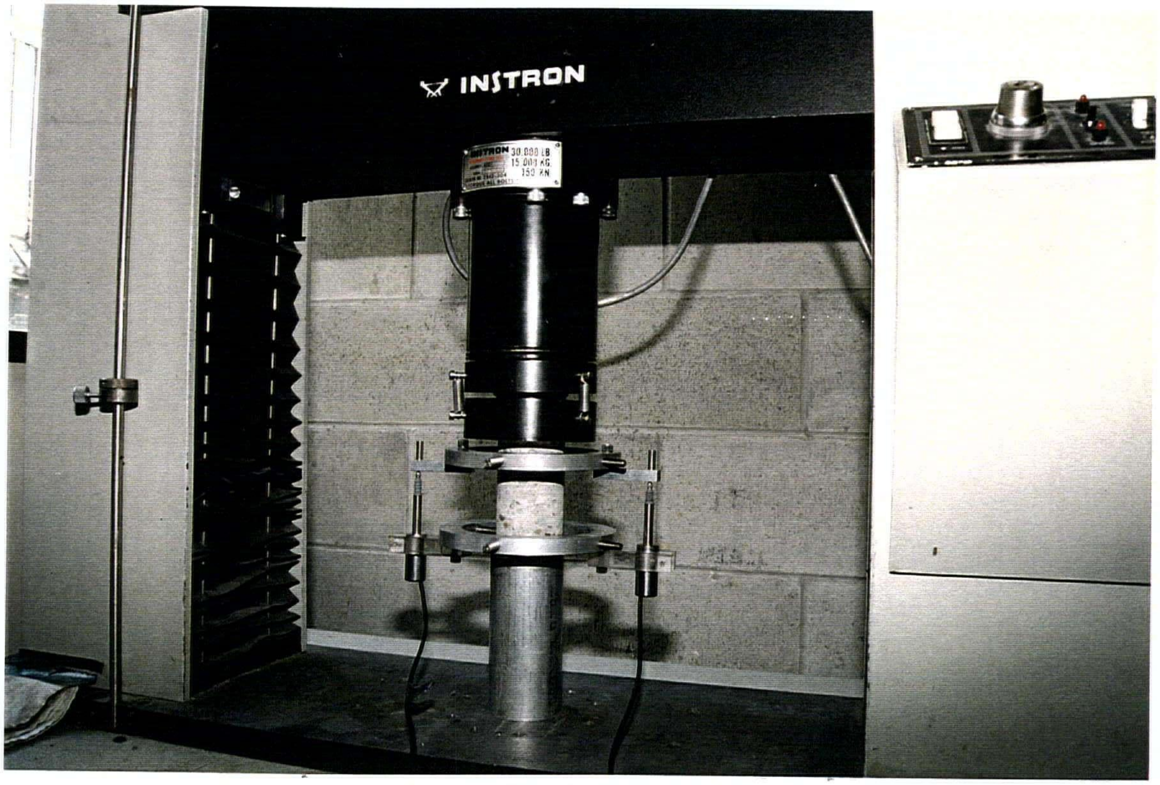


Figure 5.14 Photograph of the stress-strain test.

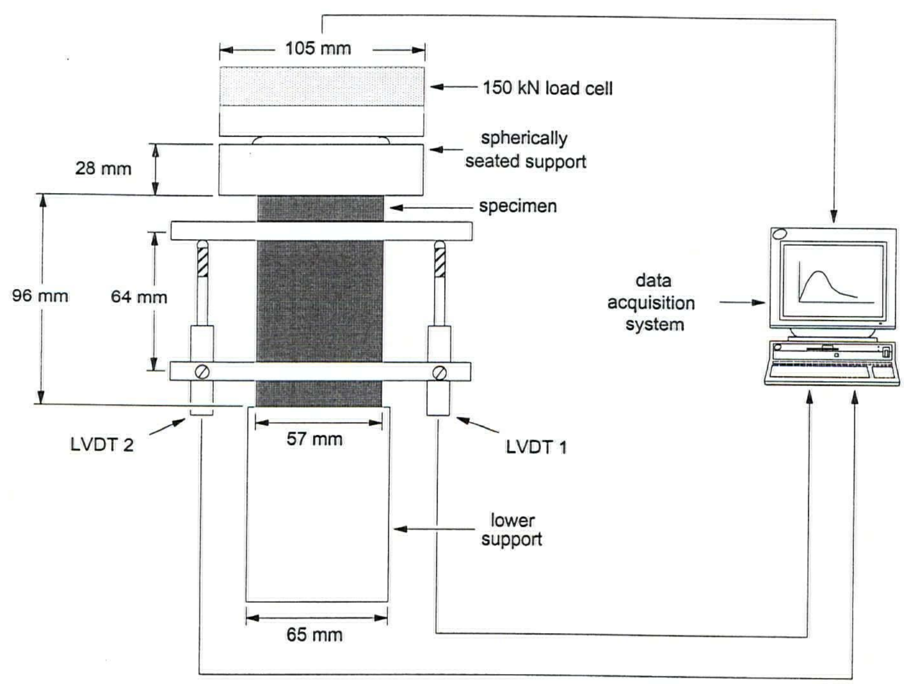


Figure 5.15 Schematic diagram of the stress-strain test.

was also required. Therefore, the specimens were first loaded at rate of about 0.25 MPa/s up to nearly 70-80% of the peak load and the rate was then changed to a cross-head displacement rate of 0.01 mm/min to obtain the rest of the curve.

For certain virgin aggregate specimens with strengths exceeding the load capacity of the Instron, the specimens were tested in a 60,000 lb hydraulically controlled testing machine (Baldwin). The specimen was instrumented in the same way as in the Instron and the loading sequence was kept as close as possible to specimens tested in the other machine.

From these stress-strain tests, the following material characteristics were evaluated:

1) stability of system;
2) compressive strength from the peak stress;
3) strain at peak stress;
4) modulus of elasticity according to ASTM C469;
5) ratio of the secant modulus to the modulus of elasticity;
6) toughness evaluation, as described in detail in Section 6.4.

5.5.2 Splitting Tensile Tests

Splitting tensile tests were also performed on cores 57 mm diameter and 96 mm high taken from the panels. The tests were carried out in accordance with the requirements of ASTM C496. The tests were also done on the 150 kN floor-mounted Instron materials test system with the load being recorded by the load cell on the machine. No difficulty was encountered in carrying this test out. The test setup is shown in Figure 5.16 and schematically in Figure 5.17. From the splitting tensile tests, only the splitting loads were obtained, which were then used to calculate the splitting tensile strength of the material.

Figure 5.16 Photograph of the splitting tensile test.

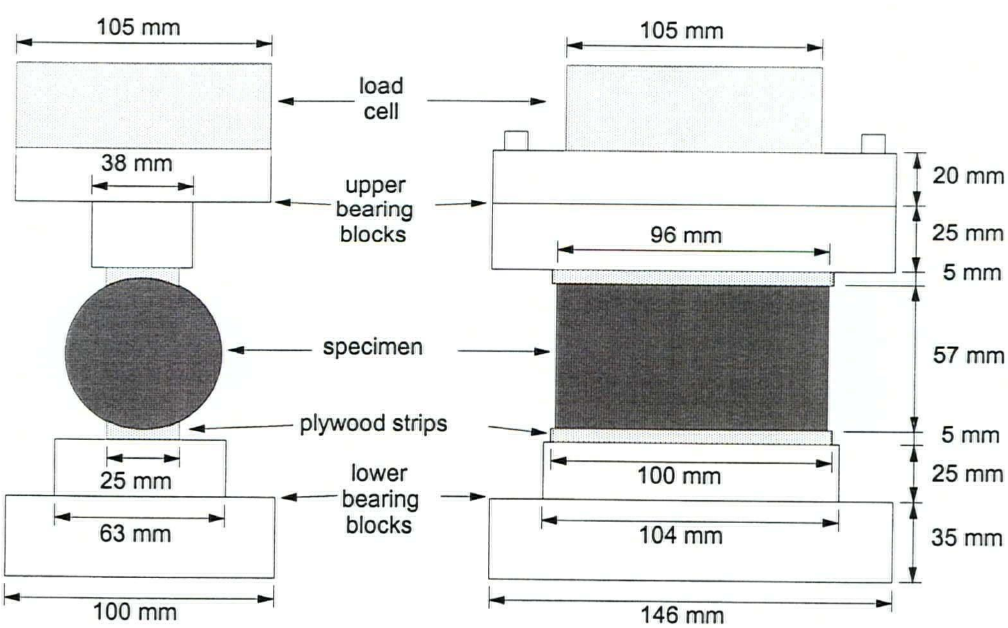


Figure 5.17 Schematic diagram of the splitting tensile test.

5.5.3 Flexural Toughness Tests

For the pilot study of fiber reinforced shotcrete, flexural toughness tests were performed on beam specimens with 100 mm x 100 mm x 350 mm dimensions, sawn from the panels. The tests were carried out in accordance with ASTM C1018-94b. They were also done on the 150 kN floor-mounted Instron materials test system with the load being recorded by the load cell on the machine and the displacement being recorded by two LVDT's mounted on either side of the specimen. Since, during flexural toughness tests, there is settlement at the supports, a yoke was installed around the specimens to allow the measurement of true midspan deflections. The signals from the load cell and the LVDT's were sent to a data acquisition system and recorded at 1 Hz. This setup is shown in Figure 5.18 and schematically in Figure 5.19.

From these tests, the following material properties were evaluated:

1) flexural strength (modulus of rupture) of the specimens;
2) flexural toughness coefficients according to ASTM C1018 indices and to the JSCE-SF4 toughness parameters.

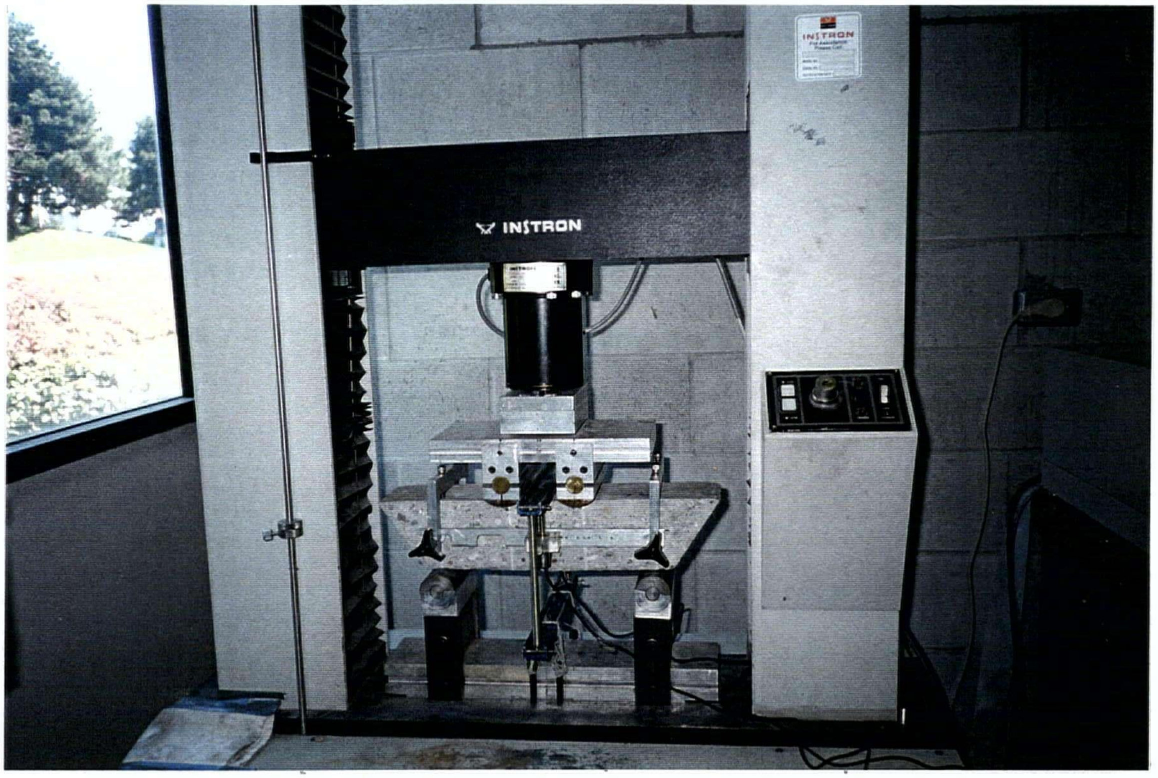


Figure 5.18 Photograph of the flexural test.

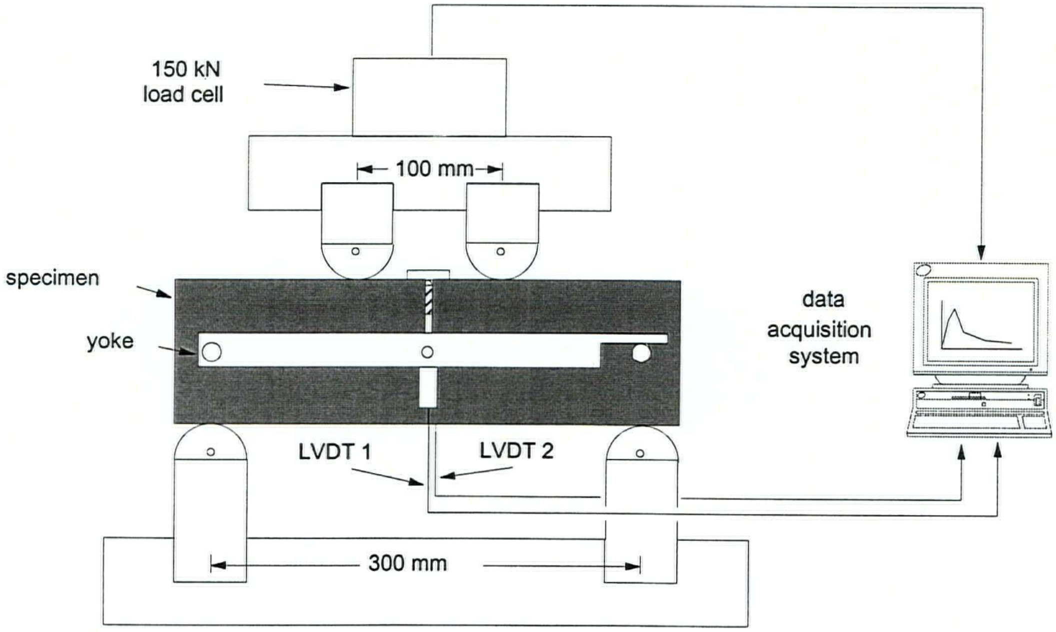


Figure 5.19 Schematic diagram of the flexural test.

6.0 RESULTS AND DISCUSSION

6.1 Stress-Strain Response

6.1.1 Results

Complete stress-strain response curves were obtained for the different mixes using the test setup and procedures described in Section 5.5.1. Stress is defined as the applied load over the initial (unloaded) cross-sectional area and strain is defined as the change in gauge length over its initial dimension. As mentioned in Section 5.5.1, the specimens were initially loaded at the rate specified by ASTM C469 up to 70-80% of the peak load, after which the rate was reduced to a cross-head displacement rate of 0.01 mm/min. This allowed for the determination of the modulus of elasticity in the standard manner as well as leading to stable loading and unloading of the specimen, thus simulating approximate closed-loop testing conditions. The results of such tests for dry-mix shotcrete, wet-mix shotcrete and cast concrete with virgin or recycled aggregates at different ages are shown in Figures 6.1 to 6.12.

Before continuing further, the importance of the definition of strains should be addressed. As concrete is loaded, increased cracking takes place in the system due to the initiation and propagation of microcracks leading to the formation of a continuous network of cracks within the loaded regions of the specimen. The cracks will, at some stage of loading (about 75-80% of the peak load), coalesce into a single continuous macrocrack running through the specimen. This phenomenon is often referred to as strain localization. Prior to this occurrence, the microcracks are almost isolated and randomly distributed, so concrete strain can be adequately used to describe the deformation state of the overall system. However, once strain localization has occurred and particularly after the peak load has been reached, deformations will take place along the macrocrack, and hence, the use of strain as a variable to describe the overall deformations becomes inappropriate. To overcome this, van Mier [63] recommends the use of total displacement, equal to specimen strain times specimen height, to describe specimen deformations after the peak load. His proposition came from observations which indicated that specimens of different heights would still have the same post-peak deformations. In the experiments carried here, the difficulties that would have occurred if strains were used to describe deformation were eliminated by making all specimens with the same height (96 mm). This way, strains in the conventional definition could be used instead of total displacement and a single stress-strain diagram could be employed for all purposes to illustrate the loading and deformation conditions in the specimens.

From Figures 6.1 to 6.12, it can be seen that, in general, the shape of stress-strain curves for recycled aggregate mixes is similar to that for virgin aggregate mixes, at least on the pre-peak side. This almost parabolic shape is one of an initial linear portion during which the material still behaves elastically followed by a gradually increasing curvature up to the peak value of stress and a descending branch known as softening. For the mixes in which unstable failure occurred, at least the stable portion of the curve is similar in shape to its counterpart aggregate type specimens. This trend holds for either dry-mix shotcrete, wet-mix shotcrete or cast concrete, at any age. Therefore, it may be implied that the processes of crack evolution with increasing load in the systems are similar regardless of the aggregate type. The only difference that recycled aggregates impart to the system is that this cracking occurs at a much earlier stage and evolves much more rapidly, leading to a lower concrete or shotcrete strength. This observation was also made by Henrichsen and Jensen [43] who then proposed that structures made from recycled aggregates can therefore be designed according to the theory of plasticity just like structures made with ordinary concrete.

6.1.2 Pre-Peak Region

It is evident from Figures 6.1 to 6.12 that mixes containing recycled aggregates undergo significant nonlinearities and their stress-strain curves display a higher curvature in the pre-peak regime. Such behaviour leads to the lower peak loads compared to mixes made with virgin aggregates.

In normal concrete, stress-strain curves portray pre-peak nonlinearities due to several main reasons. These are [54, 64]:

1) The composite nature of concrete. While the cement paste and the aggregate may be linear in nature, their combined response as a composite leads to inelastic behaviour.
2) The nature of the cement-aggregate bond. A reduction in bond strength leads to an increase in the nonlinearity of the stress-strain curve.
3) Creep of the cement paste. Creeping of the paste allows for additional deformation at sustained loads.

In the case of recycled aggregate concrete, however, each of the above factors is substantially amplified to such extent that the overall curvature in the response is much more pronounced. Considering the first factor, it may be pointed out that the moduli of elasticity of recycled aggregates as well as that of a matrix containing fine recycled material are respectively lower than the moduli of elasticity of virgin aggregates and that of a matrix with fine virgin material. Although it may not be certain how these lower modulus of elasticity values affect the degree of curvature in the resulting composite material, they do render the composite material with a lower overall modulus of elasticity at any stage in the stress-strain response. As a consequence, for every strain increment, the stress increment is much smaller for recycled aggregate mixes compared to virgin aggregates.

This concept of a composite material response would be the main factor for the observed nonlinearities only if perfect bonding existed between the matrix and the aggregates. This is, however, not the case with concrete, thus leading to the second factor listed above. It is a well-known fact that the matrix-aggregate bond in recycled aggregate concretes is much weaker than the corresponding bond in virgin aggregate mixes. As a result, much of the inferiority in mechanical properties of recycled aggregate concrete has been attributed to the weak bonds that exist in such systems. In addition to commonly occurring differential shrinkage cracking, thermal cracking and bleeding pockets that occur at interfaces in ordinary concrete, other factors exist which can severely affect bonding in recycled aggregate concrete. These are as follows:

- The presence of a large amount of dust in recycled aggregates. This dust actually forms coatings around the aggregates, effectively preventing any extensive bonding (chemical or mechanical) between the paste and the aggregates. Cracks can therefore propagate at these interfaces without much resistance.
- A lack of good bonding between the new matrix and the old matrix. Cement mortar cannot adhere to an old mortar as strongly as it would to aggregates.
- A lack of good bonding between the new matrix and contaminants present. Cement mortar cannot adhere strongly to most contaminants especially if these are soft, powdery, fibrous or greasy.
- The reduced potential for any beneficial cement paste-aggregate reaction to occur. In normal concretes, a slight chemical reaction takes place between certain aggregate types and the cement paste leading to the formation of a duplex film. This layer has a considerable strength compared to zones farther away from the actual interface [65]. In recycled aggregates, however, there is a good possibility that any such reactions would have previously already occurred. The result is that the interfacial region starts as a weak material with large oriented calcium hydroxide crystals and ettringite right at the actual matrix-aggregate interface and continues for the length of the interfacial zone.
- The higher porosity that exists between the new matrix with the old matrix or with the contaminants. This porosity is the source for most of the crack initiation and propagation that leads to the breakdown of the structure.

- The increased potential for any adverse chemical reaction to occur. Recycled aggregates which have previously been exposed to harsh environments may have the possibility of continuing such reactions in the new concrete. These reactions could be caused by sulphate, carbon dioxide, chlorides, etc. At the same time, other demolition contaminants can also produce additional detrimental reactions causing more cracking in the system, particularly at the weaker interface.

As a consequence of the above influences, it is easy to conceive that the formation of a good, strong bond between cement matrix and recycled aggregates is difficult to achieve. This results in a greater density of initial flaws at the interface which can quickly propagate even at low values of stress causing large nonlinearities in the system and premature failure. It would, therefore, seem obvious that any major improvements in concrete recycling technology would have to target the attainment of better bond. One such suggestion has been made by Ishai [66] who suggested investigating the feasibility of introducing a thin layer of a thermosetting adhesive (epoxy) between the coarse concrete rubble and the fresh matrix as a means of improving the bond. However, because of the number of different reasons affecting this bond, it is not a straight-forward task to develop a single solution that could solve all these problems, and it would be very costly.

The third reason for the nonlinearities in concrete stress-strain curves as mentioned before is due to the creep of the cement paste under applied loads. It has been found that the creep of recycled aggregate concrete is in the range of 25-60% higher than the creep of normal aggregate concrete [10, 47, 48]. Therefore, if creep induces stress-strain nonlinearities, recycled aggregate concrete is bound to creep more and to be more nonlinear than virgin aggregate concrete.

From all the above reasons, it can be seen why recycled aggregate concrete increases its stress-strain nonlinearity much more rapidly and earlier than virgin aggregate concrete . The system undergoes rapid deterioration eventually leading to failure at lower values of stresses. The above factors are independent of the specimen-making process or the age; therefore, this pattern in the stress-strain curve can be seen in all of Figures 6.1 through 6.12.

6.1.3 Post-Peak Region

The post-peak region of a stress-strain curve can only be obtained in a testing machine that is stiff enough to allow for the stable softening of the concrete specimen or in a closed-loop testing machine. The tests were carried out on the Instron materials test system which was sufficiently stiff to prove the stability differences between samples containing recycled aggregates and those containing virgin ones. As it can be seen from Figures 6.1 to 6.12, the majority of the specimens containing recycled aggregates, even up to some at 96 days of age, failed in a stable manner as indicated by a complete stress-strain diagram including a softening branch. On the other hand, virgin aggregate specimens started failing in an unstable manner at ages as early as 7 days, as indicated by a sudden, sharp drop in the load carrying capacity. This trend applied equally to all three materials: dry-mix shotcrete, wet-mix shotcrete and cast concrete.

The fact that recycled aggregate specimens, made through either process, depict this kind of stability in contrast to virgin aggregate specimens proves an important characteristic of systems containing recycled aggregates: such systems have a better energy absorbing capacity rendering them more ductile than virgin aggregate systems. This additional energy absorbing capability results as a consequence of the extensive energy dissipating mechanisms that exists in recycled aggregate systems. As mentioned before, due to a number of factors, such systems embody substantial quantities of weak points either throughout the matrix or at the interfacial regions. These weak points, under applied loads, form large networks of microcracks which propagate without much resistance. The propagation of these microcracks with increasing load, along with the fracture of soft particles and the slippage of weakly bonded parts provide a number of mechanisms to dissipate the energy that would have otherwise been stored up as strain energy in the machine or in the specimen. Hence, once the peak load has been reached and specimen unloading occurs, there is not enough stored elastic energy to cause unstable crack growth, and the specimen undergoes progressive deterioration leading to the softening branch. In some recycled aggregate specimens, such softening can be observed even at 96 days of age, which implies that, as the specimen ages, such internal weaknesses will continue to exist either as ones that never got stronger or as new ones resulting from detrimental chemical reactions.

From the above results, it can be seen that dry-mix shotcrete, wet-mix shotcrete or cast concrete, which contain recycled aggregates have the ability to absorb large amounts of energy. Failure occurs in a very stable and gradual manner resulting in a material that is far more ductile than the failure which occurs with mixes made with virgin aggregates. Therefore, mixes using recycled aggregates may be considered for applications in which a large ductility and warning of pending failure are required from the material. The quantification of this energy absorption will be discussed in Section 6.4.

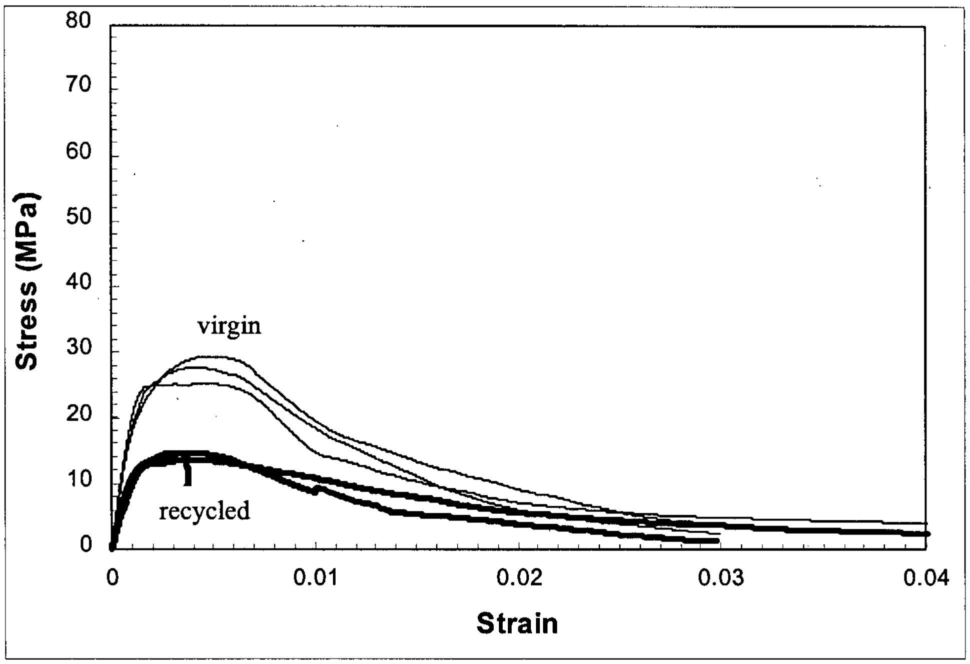


Figure 6.1 Stress-strain response of dry-mix shotcrete with virgin or recycled aggregates at 3 days.

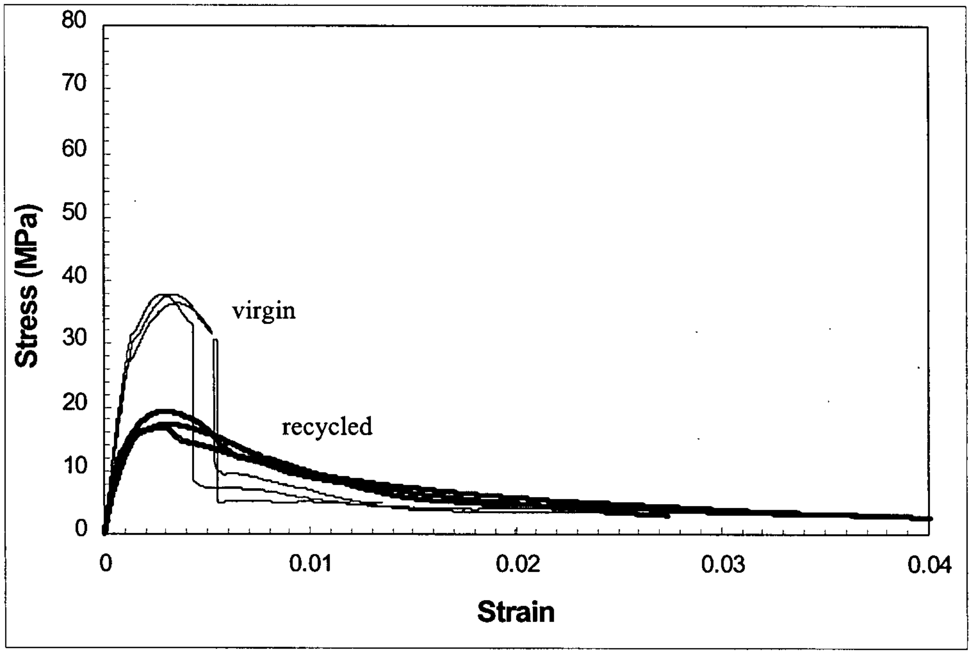


Figure 6.2 Stress-strain response of dry-mix shotcrete with virgin or recycled aggregates at 7 days.

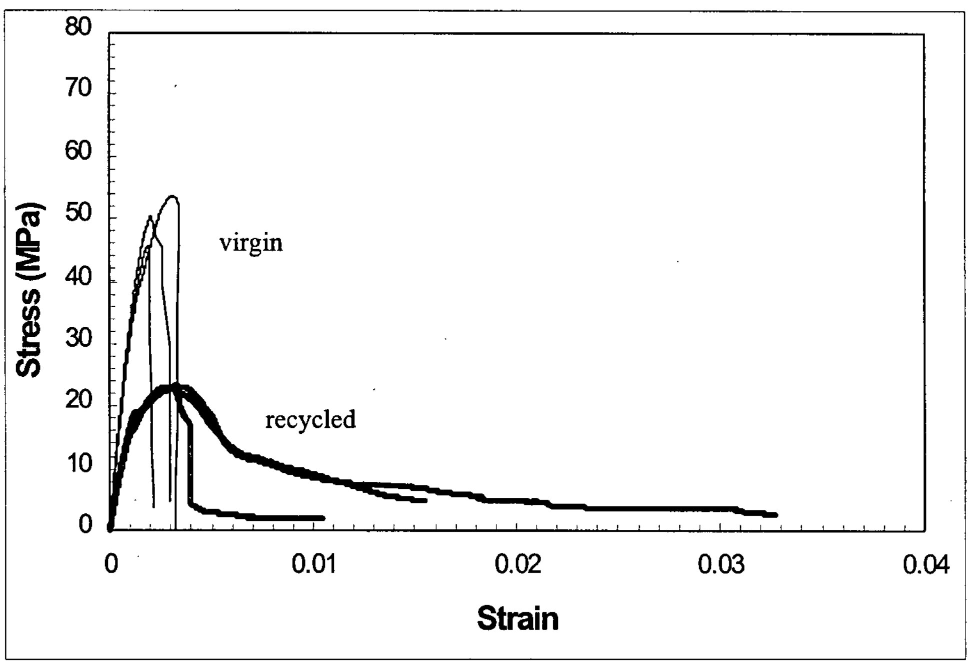


Figure 6.3 Stress-strain response of dry-mix shotcrete with virgin or recycled aggregates at 28 days.

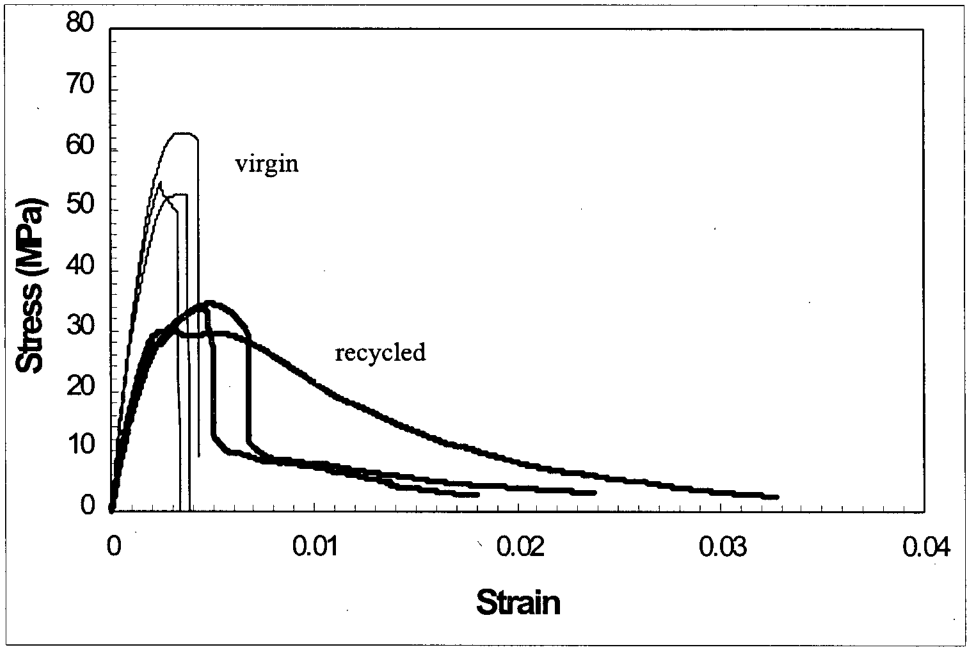


Figure 6.4 Stress-strain response of dry-mix shotcrete with virgin or recycled aggregates at 96 days.

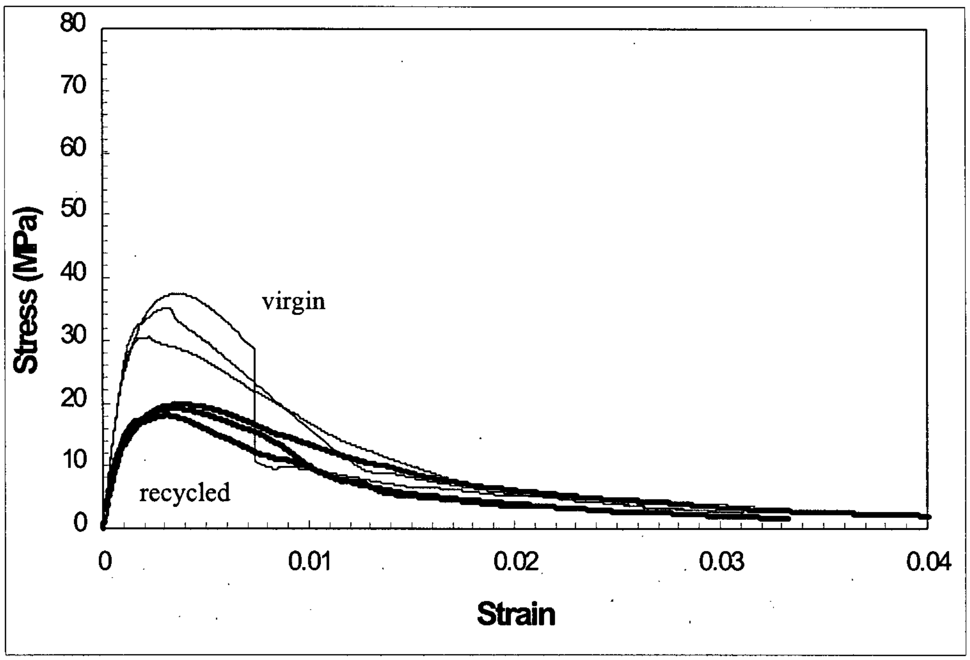


Figure 6.5 Stress-strain response of wet-mix shotcrete with virgin or recycled aggregates at 3 days.

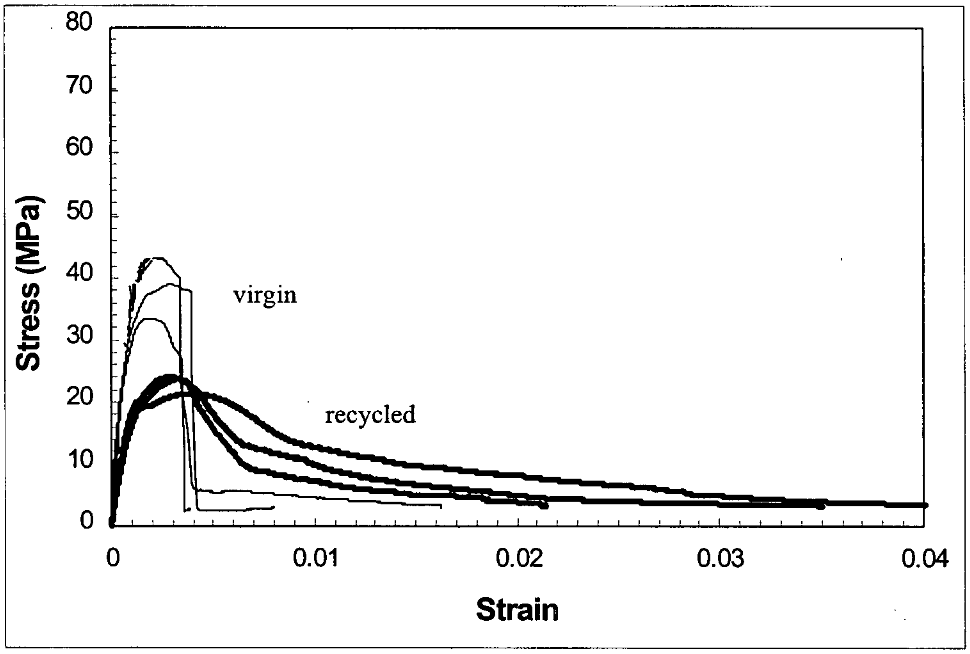


Figure 6.6 Stress-strain response of wet-mix shotcrete with virgin or recycled aggregates at 7 days.

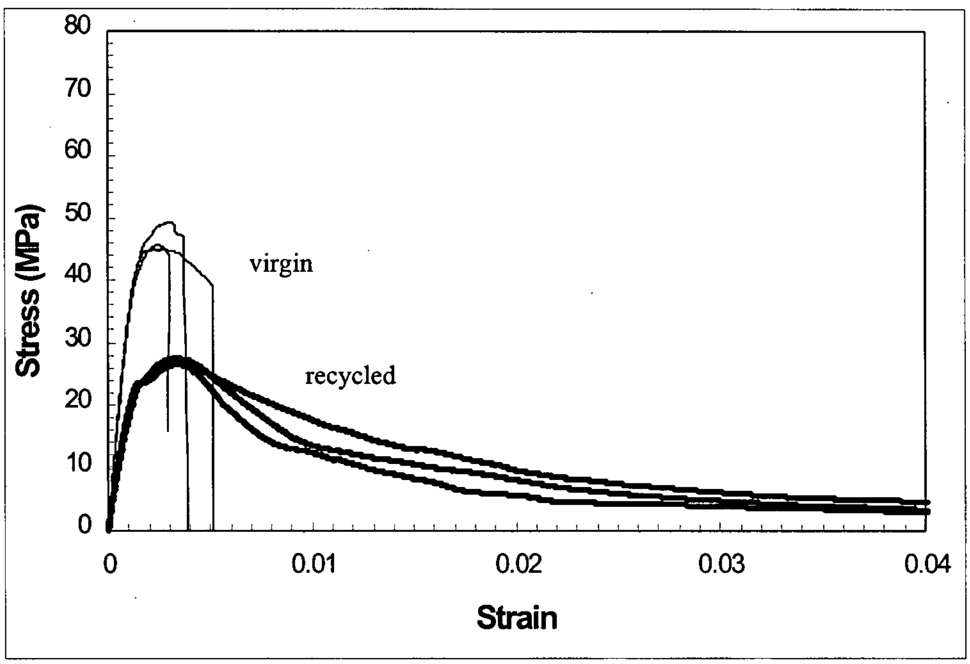


Figure 6.7 Stress-strain response of wet-mix shotcrete with virgin or recycled aggregates at 28 days.

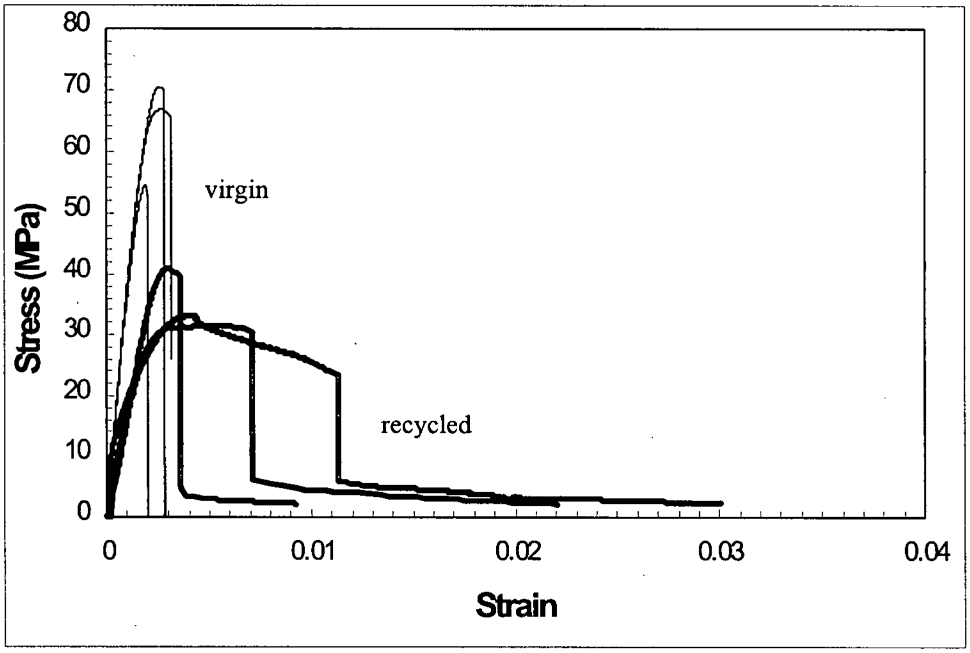


Figure 6.8 Stress-strain response of wet-mix shotcrete with virgin or recycled aggregates at 96 days.

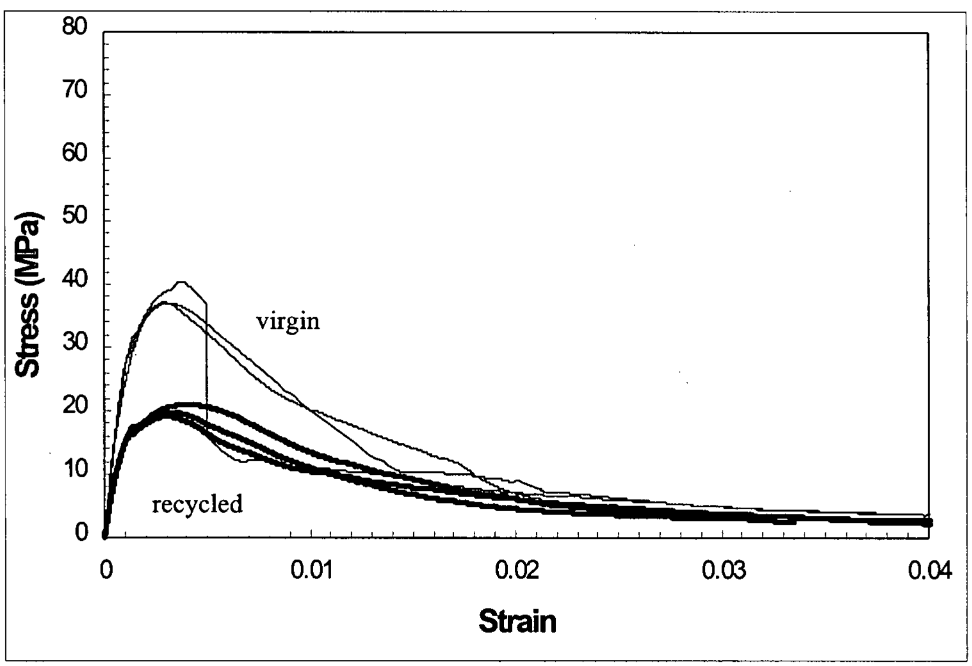


Figure 6.9 Stress-strain response of cast concrete with virgin or recycled aggregates at 3 days.

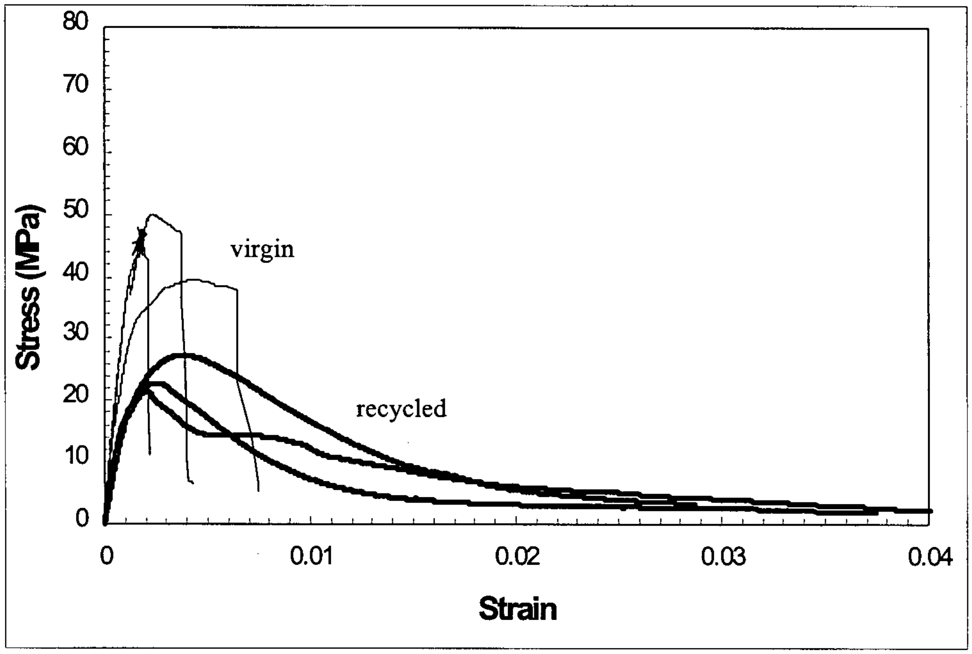


Figure 6.10 Stress-strain response of cast concrete with virgin or recycled aggregates at 7 days.

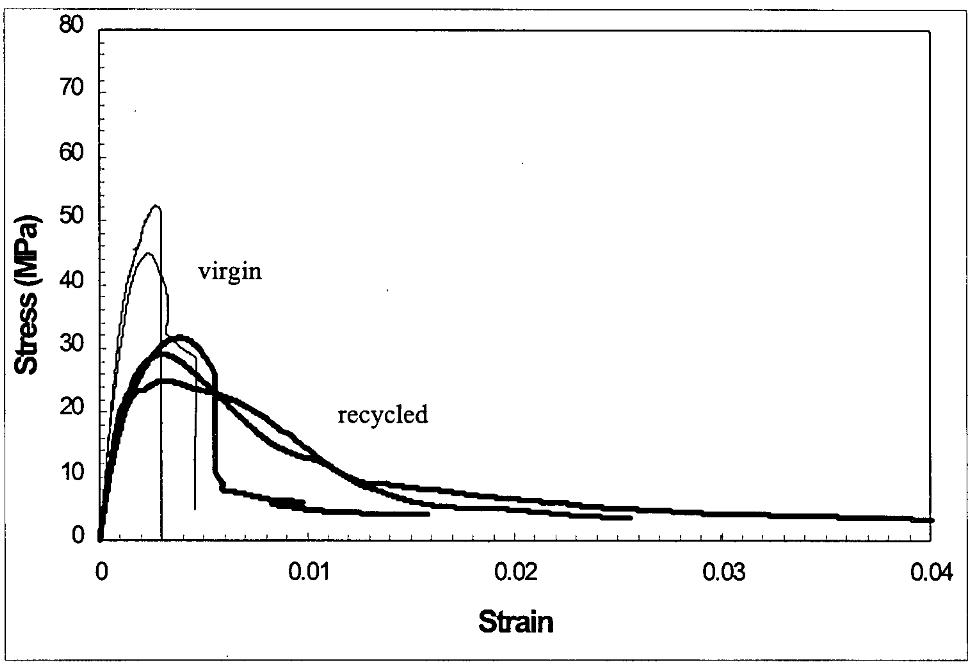


Figure 6.11 Stress-strain response of cast concrete with virgin or recycled aggregates at 28 days.

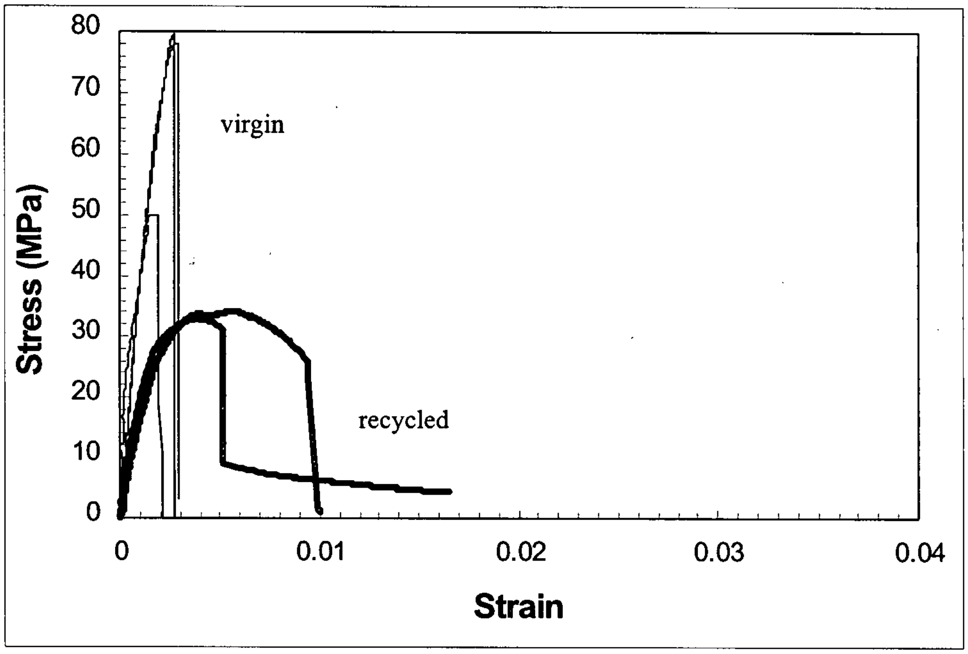


Figure 6.12 Stress-strain response of cast concrete with virgin or recycled aggregates at 96 days.

6.2 Compressive Strength

6.2.1 Test Results and Discussion

The compressive strength of concrete is the most widely used index, whether direct or inverse, of most of the other properties of practical significance. Because, in general, many of these other properties are to a certain extent related to strength and since the compressive strength is also a relatively easy property to determine, it is very common to use this parameter to assess the quality of the concrete. Thus, most of the research carried out on recycled aggregate concrete has focused on determining its compressive strength and using it to evaluate the suitability and performance of the new concrete.

From the stress-strain curves in compression presented in Section 6.1, the compressive strength was simply taken as the peak value of stress. This value is in reality not purely a material property. Just like the stress-strain curve and its shape, it will always reflect the total specimen-machine interactive behaviour. So, in order to investigate the effects of material composition, similar, if not identical, testing conditions had to be maintained among all the tests. The results of these tests for the compressive strength of the mixes at the ages examined in this program are shown in Figure 6.13.

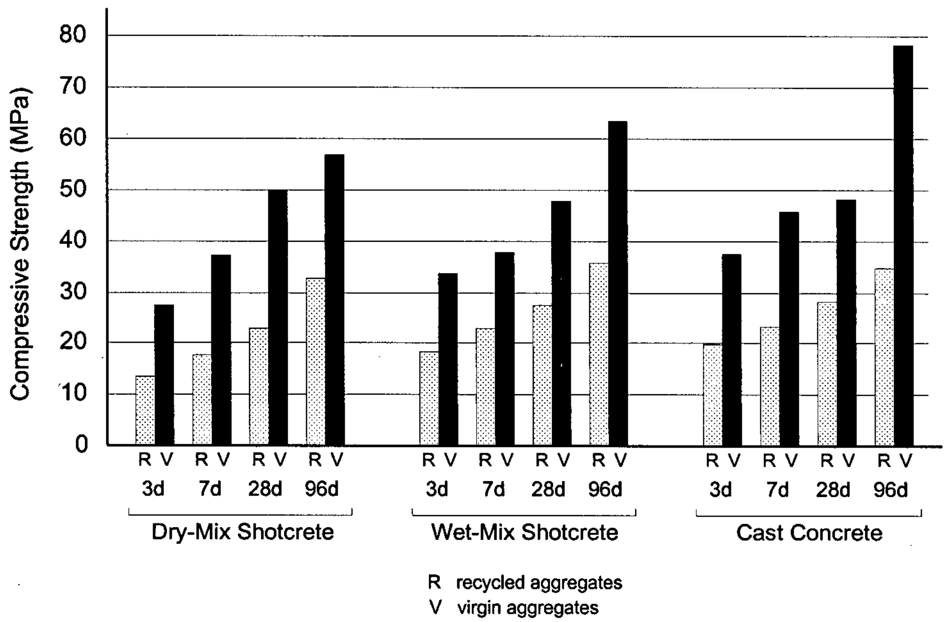


Figure 6.13 Compressive strengths of the mixes investigated.

As expected, mixes with recycled aggregates yield strengths that are lower than those obtained with mixes containing virgin aggregates. This has been the conclusion arrived at by most other researchers, and it is not surprising to witness it again, considering that crushed waste concrete is an impure substance that is certainly not as ideal as conventional aggregates in terms of possessing ideal aggregate properties. A lower compressive strength would, in the traditional sense, imply that many other properties of the concrete are also inferior and probably undesirable for fulfilling particular purposes. However, this remains to be proven because, in many applications, trade-offs between properties are accepted and the demand placed on certain properties may not be as strict.

In order to understand why there is such a decrease in the compressive strength of recycled aggregate specimens as opposed to those with virgin aggregates, it is helpful to consider the internal structure of the specimens and the mechanisms that take place during failure. The amount of sustainable load in concrete is governed by the extent to which microcracks grow inside the system and the point at which this growth becomes critical and unstable. As mentioned in Section 6.1, recycled aggregate concretes are bound to have more initial defects and flaws than virgin aggregate concretes due to a number of reasons. The main cause is most likely attributable to the large amounts of dust in the system, particularly after dry-mixing. This powder-like material, besides preventing the formation of a strong, coherent matrix, also reduces the potential for the attainment of a similarly strong bond between the matrix and the aggregates. This dust consists mainly of pulverized old matrix containing large quantities of calcium hydroxide and pulverized soft material like brick, gypsum, etc. So, in effect, no effective or beneficial chemical interaction can take place between the new matrix and zones containing dust, leaving behind in the system large quantities of pockets where cracks can either initiate or propagate through without much difficulty. Apart from the dust, the lack of good attachment between the new matrix and the old mortar and between the new matrix with the contaminants also leads to a higher possibility of cracks forming prematurely at low levels of load. Moreover, some adverse chemical reactions like that with gypsum might have caused cracks to form even at an early age in the concrete. Due to all these reasons, a system with recycled aggregates can be perceived as one that is already damaged and full of defects from the beginning prior to the application of any loads.

When a load is then applied to the system, the initial defects can grow in different directions and under different modes (opening, sliding, tearing). Stress concentrations at the crack tips promote the growth and the eventual networking of microcracks in the system. On the other hand, the accumulation of resistances in the overall system provided by the adhesion and cohesion forces needed to be overcome at each advancing crack causes an increased demand in the applied load. Thus, load can be continuously applied until the peak load is reached, at which point a macrocrack, formed by the coalition of microcracks, reaches its critical length and continues to propagate without a further increase in load. In the case of recycled aggregate concrete, its resistance to propagating cracks can only increase to a limited extent compared to virgin aggregate concrete. This is because it does not take very much load to propagate the internal cracks in the system with recycled aggregates. Besides having a larger number of initial flaws from which crack coalition can rapidly occur, the presence of a softer matrix and softer particles poses less resistance to advancing cracks in the system. The result is a quickly deteriorating system where cracks do not find much resistance in their buildup. This is indicated by a rapid nonlinearity in the stress-strain behaviour eventually leading to a lower value of the peak load. Such lower resistance to crack growth in recycled aggregate concrete leads to the notion that this material, from a fracture mechanics standpoint, has a lower value of the fracture toughness parameter, K_C [80].

Considering the possible causes for a lower compressive strength at a higher scale of observation, some researchers have pointed out that it is the strength of the recycled aggregates that will determine the eventual strength of the recycled aggregate concrete [20, 24]. From the experimental results, this proposition seems plausible, considering the fact that most of the foreign substances which are not original aggregates, such as old mortar, brick chunks, asphalt pieces, and glass, are weaker compared to normal aggregates. This might explain the lower strengths of the new concrete. However, as pointed by Johnston [67], it is not so much the compressive strength, but rather, the elastic modulus of the aggregates that determines the concrete strength, particularly for aggregates with high crushing strength. This is because the stiffness of the aggregates affects the energy release capacities which in turn affects the propagation and coalition of cracks. Applied to recycled aggregate concrete, both propositions (aggregate strength and aggregate elastic modulus) seem logical. Many particles in the stream of waste concrete have low crushing values, and at the same time, have low elastic moduli. So, it may be a combination of these two factors that causes the lower values of compressive strength in recycled aggregate mixes.

Having now considered the possible reasons for lower compressive strength in recycled aggregate mixes, it is worthwhile to consider another parameter which is the percentage reduction in strength from virgin aggregate mixes. As mentioned in Section 3.5.1, this parameter has been reported more often and is equally important as compressive strength itself. It gives an indication of the loss in potentially achievable strength when virgin aggregates are replaced by recycled aggregates at a given proportion. For the different mixes at different ages, these values are shown in Table 6.1. Their converse values, the percentage of the strength achieved with virgin aggregates, are plotted in Figure 6.14. These values correspond to a mix where both, the coarse and the fine, fractions of the aggregate are replaced with recycled aggregates.

Table 6.1 Percentage reduction in strength for mixes with recycled aggregates.

	Percentage Reduction (%)		
Age	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete
3 days	48	45	48
7 days	53	40	48
28 days	54	42	41
96 days	43	45	56

These values indicate that, if the entire spectrum of aggregate sizes were replaced with recycled aggregates, about 40-56% of the potentially achievable compressive strength would be lost. It remains to be seen whether or not other properties of the concrete are also affected to a similar extent.

Finally, as per ASTM C39, the failure pattern of the specimens was also observed. Nearly all specimens fractured in the shear mode with the formation of a cone. This was however deemed to be more due to specimen-machine interaction than a material property. It was interesting to note that the broken recycled aggregate specimens were very powdery and the pieces fell apart very easily, whereas the broken virgin aggregate

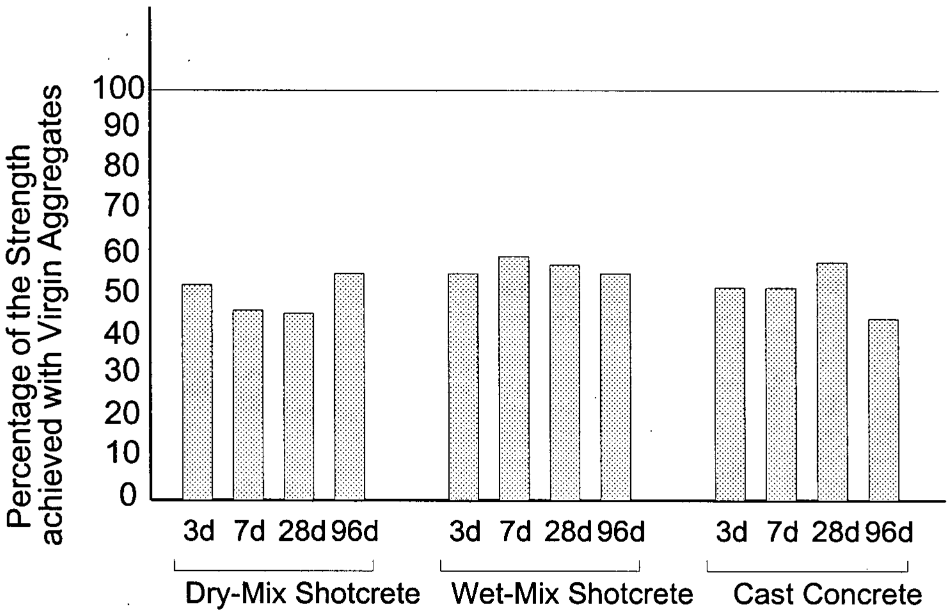


Figure 6.14 Percentage of the strength attained with virgin aggregates for the different mixes.

specimens remained as a few solid chunks. This is again indicative of the high fraction of fine material which hindered the formation of bonds within the system and remained there as unreactive particles.

6.2.2 Comparison with Other Works

In the previous section, the compressive strength results were presented along with the possible reasons why differences between recycled and virgin aggregate mixes are observed. However, rather than restricting the focus to this research alone, it is worthwhile to look on a larger scale and see how these results actually compare with other published data. This would give an indication of whether any actual changes in the technology have been achieved by the methods utilized in this program. Due to the different mixes and mix proportions used in this program compared to other investigations, again, the useful parameter to consider is the percentage reduction in strength. The results for the cast concrete mixes are mainly considered here because all the collected literature deals with cast concrete. At the same time, these results are compared with concretes for which the entire aggregate fraction, not just the coarse fraction, is replaced with recycled aggregates.

In Section 3.5.1, it was mentioned that, for the literature collected, the percent reduction in strength for concretes where the entire range of aggregate sizes is recycled lies in the range of 20-40%, with one result up to 80%. Most of these tests were carried out at 28 days. From the above experimental results, the cast concrete strength at 28 days has a 41% strength reduction which is in agreement with the other literature. This value changes at other ages but cannot be directly compared with the published data because the latter were carried out mostly at 28 days and concrete is a system that changes with time. For interest purposes, the corresponding numbers for shotcrete can also be looked at. Wet-mix shotcrete appears to have similar reductions in strength to the published data for cast concrete, while dry-mix shotcrete shows a larger decrease in strength. Thus, it may be speculated that wet-mix shotcrete is affected by the inclusion of recycled aggregates in a similar way to cast concrete, either in this research or in the previous investigations; but for dry-mix shotcrete, other processes take place which may lead to higher strength reductions.

One of the ways in which the published literature was examined in Section 3.5.1 was to compute with time the strength of the recycled aggregate mixes as a percentage of the corresponding virgin aggregate mixes. The same figure (Figure 3.4) is plotted and the results for this experiment are also included. This is shown in Figure 6.15. It can be seen that not much improvement has been made over time. The percentage reductions in strength still remain quite variable indicating that there is still not a definite method that will prevent such high losses in strength when recycled aggregates are employed. The large variability can also be due to the fact that, in this set of results considered, there is a significant amount of fine material in the system. As Hansen had suggested, this fine material gives rise to a number of problems and brings larger uncertainties to the quality of the resulting mixes.

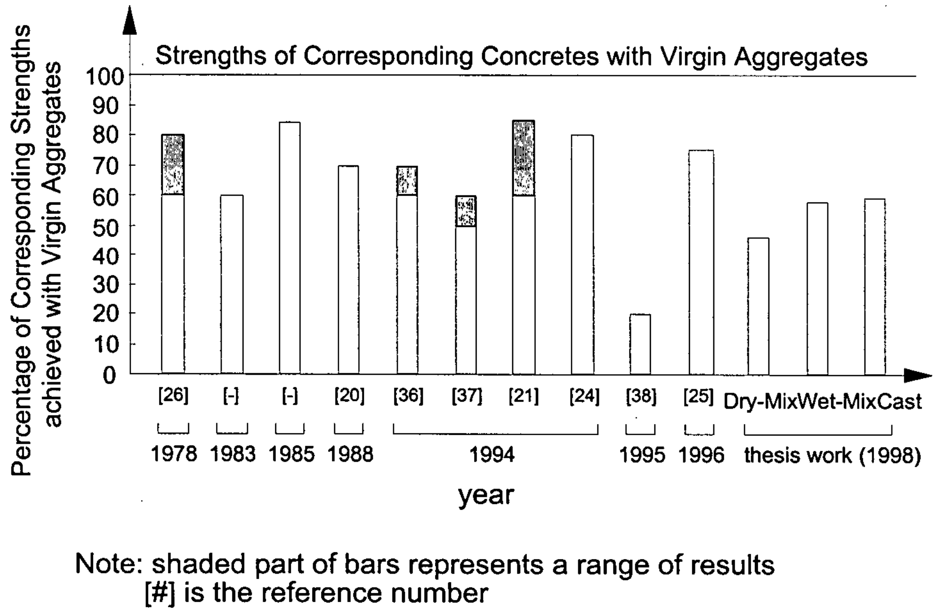


Figure 6.15 Strengths of recycled aggregate concrete as percentages of the strengths of corresponding virgin aggregate concrete over time (thesis results included).

Another way in which the published literature was observed in Section 3.5.1 was to look at the water-cement ratio law. In this research, the effective water-cement ratios for the wet mixes were maintained at 0.32, and for the dry-mix shotcrete, it was estimated at 0.42. Figure 6.16 shows the strength to water-cement ratio relationship (from Figure 3.5, with only the data for mixes where all aggregate sizes were replaced) with the new values obtained in the experiments. It is seen that the mixes containing recycled aggregates in this research deviated quite significantly from the established water-cement ratio law for virgin aggregates. However, some of the published data also deviate by a similar margin. Again, although it may be speculated that mixes containing recycled coarse and fine aggregates give a water-cement ratio law with a trend similar to that of virgin aggregates, this proposition cannot be verified given the large variability in the results. More data on this subject would be required.

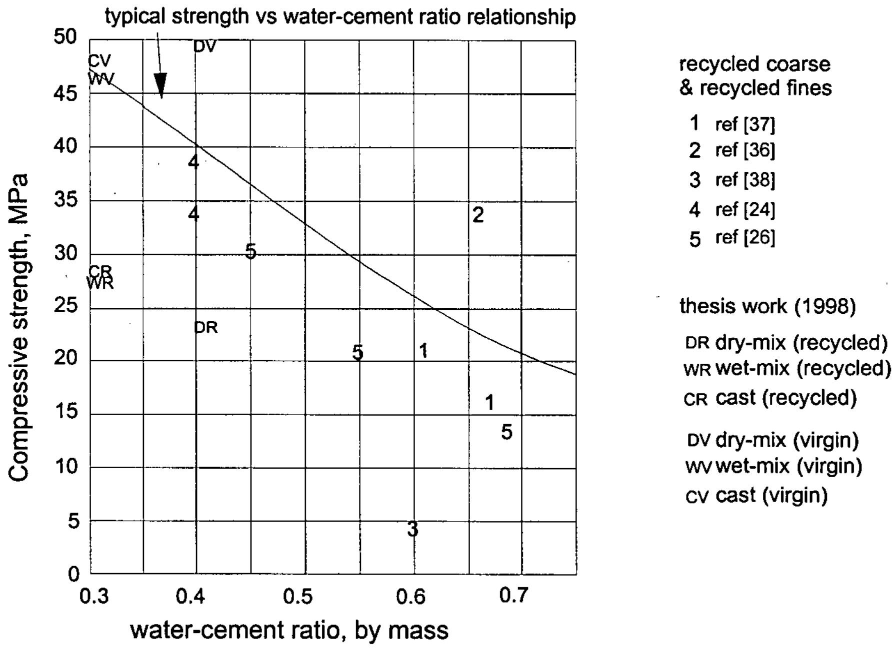


Figure 6.16 Variation of the compressive strengths of concrete with water-cement ratio for virgin and for recycled aggregate concretes.

The above considerations, with respect to the published literature, suggest that the methods employed in this research did not produce any particular improvement in the use of recycled aggregates. The percentage reductions are still about the same as they were years ago. It is very difficult to assign the reason for such reductions to a particular factor. There are many different factors interacting with each other, ranging from the amounts of fines in the system to their composition to the existence of certain foreign matter, that leads to high levels of variability in the results. At the same time, the recycled aggregates that were used for a certain project on a certain date are very likely to differ substantially from those used on another project on another date. It is more a question of variability in the quality of the material which leads to the scatter of results. However, at least, having a knowledge of which possible factors can contribute to this deterioration in performance gives researchers the advantage to target specific solutions for the problem.

6.2.3 Comparison Among Processes

In the research program, the three different production processes were examined: dry-mix shotcrete, wet-mix shotcrete and cast concrete. Since these processes are executed in dissimilar manners and under different principles, it is expected to observe differences in the obtained results. This is particularly true when comparing shotcrete and cast concrete because, in the former process, the rheological properties and the way in which the material is compacted are very different to those in cast concrete. Cast concretes would invariably result in lower plastic yield strengths [68]. Even between the two types of shotcrete, dry-mix and wet-mix, there can be differences due to the mode in which particles interact with each other. Dry-mix shotcrete's unique feature of water contacting the cement and reaching its final mixing position within a fraction of a second provides it with inherently unique workability features and properties that would be different from those achieved by the wet-mix shotcrete process [68]. When recycled aggregates are used in shotcrete, differences from the expected behaviour may occur simply because of the way in which the mixes are placed.

The compressive strengths of the mixes investigated are shown in Figure 6.13. First of all, to examine the differences that shotcrete produces over cast concrete, one can look at the strength differences between wet-mix shotcrete and cast concrete, since these two were produced with the same mix proportions. For both recycled and virgin aggregates, the cast concrete mixes yield similar, if not higher, strengths than their wet-mix counterparts at all ages. This is somewhat surprising considering that the high particle velocities in wet-mix shotcrete should achieve greater compactions, thereby leading to higher strengths. Such trends were also observed by Banthia et al. [58]. One possible reason for this unexpected result in this research could be the fact that the mixes were fairly stiff (at an effective water-cement ratio of 0.32) to begin with. With cast concrete, the table vibrator was able to produce sufficient compaction in the mix to provide a uniform and homogeneous system. However, when shotcreting, as it was mentioned in Section 5.4.3, there were slight problems with the pumping of the material through the hose. In some instances, particularly for recycled aggregates, the material passed through the hose in pulses as a result of the high angularity of the particles and the cohesiveness of the mix. The lack of a continuous stream of material could have led to inhomogeneities of the in-situ mix. At the same time, such intermittent flow of material cannot achieve the same degree of compaction as a continuous stream does. A possible way to correct this deficiency is to use air-entrainers in the shotcrete mixes. As Beaupré [69] indicated in his work, a high initial air content is a good way of reducing the yield stress of the plastic wet-mix concrete hence, enhancing its pumpability. On impact with the receiving surface, air is lost during compaction which in turn increases the yield stress of the in-situ mix and hence its "shootability", which is a measure of compaction and build-up. In this way, hose blockages and pumping difficulties may be prevented from occurring while attaining a proper in-situ mix. This was, however, not carried out in this research.

Next, to consider the effects that recycled aggregates may have upon the mixes in the different processes, one can look at the percentage reduction in strength for each case. These values have been presented in Table 6.1 and are repeated again here.

	Percentage Reduction (%)		
Age	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete
3 days	48	45	48
7 days	53	40	48
28 days	54	42	41
96 days	43	45	56

There does not appear to be any definite pattern in the variations of this parameter. Values lie in the range of 40-50% strength reduction with some values of more than 50% reduction. Since these values lie within a more or less similar range, it can be speculated that the mechanisms by which recycled aggregates deteriorate a cementitious system are similar regardless of the production process. Moreover, it is likely that such variations in this parameter are due more to the variability of the material, as expected when the coarse and fine aggregate fractions are replaced with recycled material, than to differences in the method by which they are produced.

During production, the dry-mix shotcrete process may provide an advantage over the wet-mix process in that there is not enough time for the recycled material to manifest its higher water absorption as the material reaches the receiving surface in a fraction of a second; thus, there is a less water demand and, at the same time, less workability problems. This behaviour was reflected in the identical penetration resistances obtained for the dry-mix shotcrete with virgin aggregates and the dry-mix shotcrete with recycled aggregates. On the other hand, the wet-mix shotcrete process may offer other advantages over the dry-mix process in that the initial mixing of the ingredients, including water, in a mixer produces a more uniform and coherent mixture, and any effect that the fine material may have would have been accounted for while mixing. Such is not the case for dry-mix shotcrete because the large quantities of dust and impurities in the system could, in a fraction of a second, interfere with the intimate mixing needed to take place between the injected water and the moving stream of dry materials. This may result in some unwetted zones in the mix as well as some dust pockets. In any case, the final result which is the in-situ shotcrete, for both types of processes, ends up with similar percentage reductions in strength, indicating that the net effect of incorporating recycled aggregates as aggregate replacements into the mix is nearly similar. Since this reduction in strength is also very similar to that in cast concrete, it may well be that the production processes employed do not significantly alter the influences the recycled aggregates could have. The internal structure of the hardened material and the aging factor seem to have a far greater influence.

From the above, it can be seen that, as far as the compressive strength is concerned, the use of shotcrete does not provide any improvements over cast concrete when recycled aggregates are employed. At the same time, there are no additional impairments either. It appears that the wet-mix shotcrete process offers more promise for using recycled aggregates than the dry-mix process does. This is because, apart from yielding somewhat lower percentage reductions in strength, there is more room for improvement with the addition of air-entrainers and more superplasticizer to enhance the workability of the mix. This, however, remains to be proved.

6.2.4 Strength Gain

The strength gain properties of recycled aggregate concrete have not been investigated in much greater details in previous works [10]. Some of the published literature has reported that the rate of strength development in recycled aggregate concrete is very close to that in virgin aggregate concrete, as indicated by similarly shaped curves in the strength-age relationship. It is interesting to see if a similar trend also applies to shotcrete specimens. At the same time, the strength values at later ages (96 days here) provide an idea if the long term performance of the material has been affected by the presence of such aggregates, thus indicating any potential for durability related problems.

The compressive strength results as a function of age for the 6 different mixes investigated are shown in Figure 6.17. In general, it can be seen that, up to 28 days of age, the strength gain characteristics for all mixes are fairly similar, except for the dry-mix shotcrete with virgin aggregates which deviates from the trend followed by the other mixes. This observation supports the earlier findings that the strength gain of recycled aggregate mixes is comparable to that of virgin aggregate mixes. Such similarity also applies to the different production processes, whether cast or shot. Note that, for recycled aggregates in particular, the strength gain curves for dry-mix shotcrete, wet-mix shotcrete and cast concrete are almost parallel to each other. Thus, the method used for production does not seem to affect the rate of strength development. At the same time, when comparing mixes with recycled or virgin aggregates, the type of aggregate does not have a significant effect either, at least up to 28 days of age.

At later ages, a discrepancy can be seen between recycled aggregate mixes and virgin aggregate mixes. This is because the 96 day strength for the mixes does not follow the

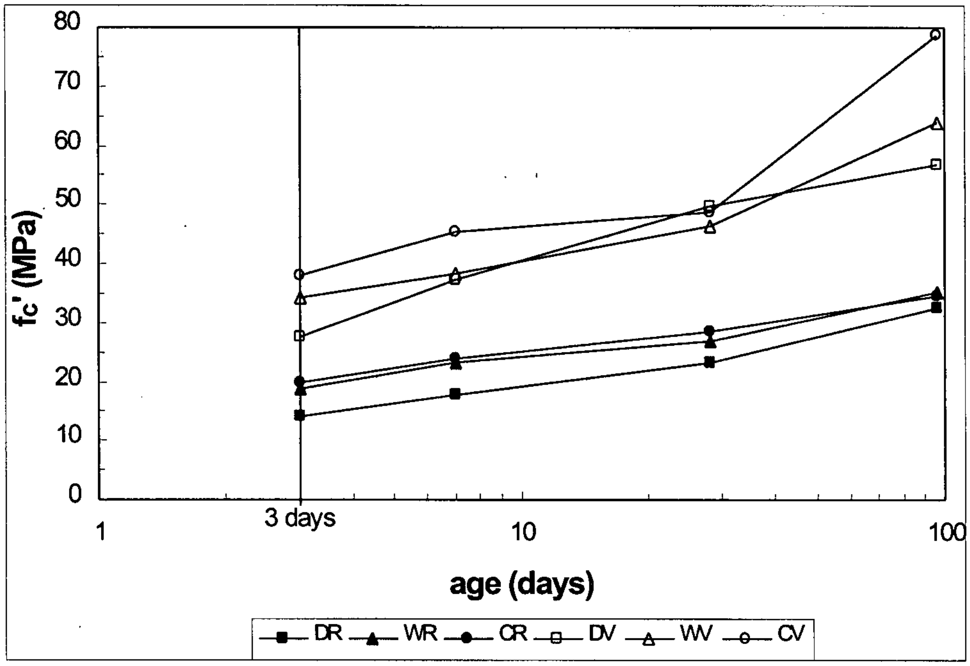


Figure 6.17 Compressive strength vs. age relationship for different mixes (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

same pattern. Strengths at this age for virgin aggregate specimens improve appreciably from the strengths at 28 days, but such is not the case for recycled aggregate specimens. In other words, recycled aggregate specimens have a limited capacity for continued strength gain past 28 days of age. One possible cause for this is the large amount of dust in the system which inhibits the potential for the system to acquire a stronger, more coherent internal structure. As mentioned in Section 6.2.1, the dust in the concrete creates spaces where the cement paste cannot penetrate, thus forming voids. At the same time, this dust interferes with the formation of good bonding between matrix and aggregates. The result is that the system can only attain a certain degree of solidity and this degree is reached very rapidly in the first 28 days of age. Another possible reason for the lack of strength development in recycled aggregate mixes past 28 days is the fact that these aggregates incorporate large amounts of contaminants, many of which are reactive. For example, gypsum, brick, glass or attacked old concrete have the potential to induce a number of deleterious chemical reactions in the new concrete. Such reactions would contribute extra flaws and deficiencies into the system which serve as crack initiators. This is actually one of the biggest challenges when using recycled aggregates since it is not possible to determine the extent to which these injurious reactions could take place. Also, it is not possible to determine exactly the specific sources of possible adverse reactions from a given batch of recycled material. As a result, the durability of recycled aggregate concrete, as well as shotcrete, becomes a matter of serious consideration. It is strongly recommended that long term tests, say at 6 months, 1 year, 2 years, etc., be carried out on recycled aggregate mixes to observe the extent to which damage can reach until, as well as whether or not there is a possibility of an actual decrease in the strength of the mix.

6.2.5 Fiber Reinforced Mixes

A pilot study was carried out as part of the research program to study the effects of fiber reinforcement in recycled aggregate shotcrete. Only the properties of wet-mix shotcrete at 28 days were investigated. The exact same mix proportions and material preparation methods as the mainstream wet-mix shotcrete were employed, with the exception that 30 mm hooked-end steel fibers at an initial content of 1.0% by volume were added. The compressive strength results for these mixes are shown in Figure 6.18.

The compressive strength values for the unreinforced mixes of this pilot study are somewhat higher than the results obtained in the foregoing sections. This is possibly due to the fact that these specimens in the pilot study were tested at the standard loading rate which is much faster than the rate at which the stress-strain tests were carried out. However, the interesting feature to be seen from these results is that fiber reinforcement increases the compressive strength of shotcrete. This observation was also made by several other researchers [70, 71, 72]. Moreover, such improvements in compressive strength occur irrespective of whether virgin aggregates or recycled aggregates were used in the mix. It could be that, at such relatively high fiber additions, there is some crack

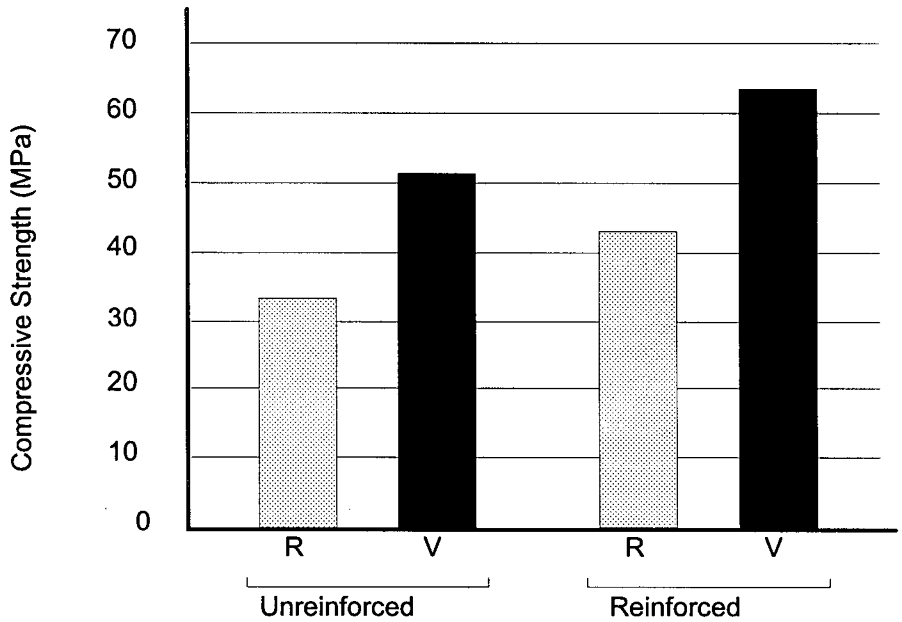


Figure 6.18 Compressive strength of fiber reinforced mixes at 28 days (R=recycled aggregates, V=virgin aggregates).

arresting mechanisms provided by the fibers which provides an increase in strength. Nevertheless, such improvements in strength are not significant to justify the use of fibers for increasing the compressive strength.

6.3 Strain at Peak Load

6.3.1 Test Results and Discussion

The strain at peak load is simply obtained as the value of strain corresponding to the peak stress in the stress-strain diagram. It represents the amount of deformation incurred within the material when it has reached its maximum load carrying capacity. In design, this value is normally taken to be in the range of 0.0030 to 0.0035, and the material is considered to have failed when it reaches this value of strain. However, this quantity could vary depending on the material composition and the mix proportions. For convenience, the strain at peak load will be called ultimate strain since, in design, it corresponds to failure.

Looking at the physical situation, when concrete is loaded, it undergoes elastic deformations as well as some deformation due to crack advancement. At low levels of stress, these cracks extensions are insignificant and also randomly distributed within the system, and strain is still an adequate parameter to describe the overall material deformation. At higher levels of stress, however, beyond about 75% of the peak stress, cracks coalesce into a major macrocrack that runs through the matrix and, perhaps, through some aggregates. This is the crack that continues to grow under applied load and becomes critical at peak stress. The formation of this crack changes the deformation state within the material as follows: (1) elastic unloading of the regions away from the crack occurs, and (2) material deformation takes place mainly along this crack. As a result, material strains cannot be well-defined at this stage.

The results for the ultimate strain for the mixes investigated are shown in Figure 6.19. It can be seen from them that recycled aggregate concrete or shotcrete has a generally larger value of ultimate strain compared to the virgin aggregate mixes. This is particularly evident at later ages when the specimens have attained a sufficiently solid and stable internal structure. The values at 28 days, which is normally the design age, lie in the range of 0.0030-0.0035 for recycled aggregates, whereas for virgin aggregates, they fall closer to 0.0025. This represents an almost 20-40% increase in this variable when recycled aggregates are used instead of virgin ones.

The above observations can be explained by the fact that recycled aggregate systems undergo much more cracking, both at microscopic or macroscopic levels, as they are loaded. This supports the material presented in Section 6.2.1 when it was mentioned that the higher number of initial flaws and the higher potential for new defects to be created while the specimen ages cause systems with recycled aggregates to experience more severe crack propagation. In that section, this internal behaviour was seen from the

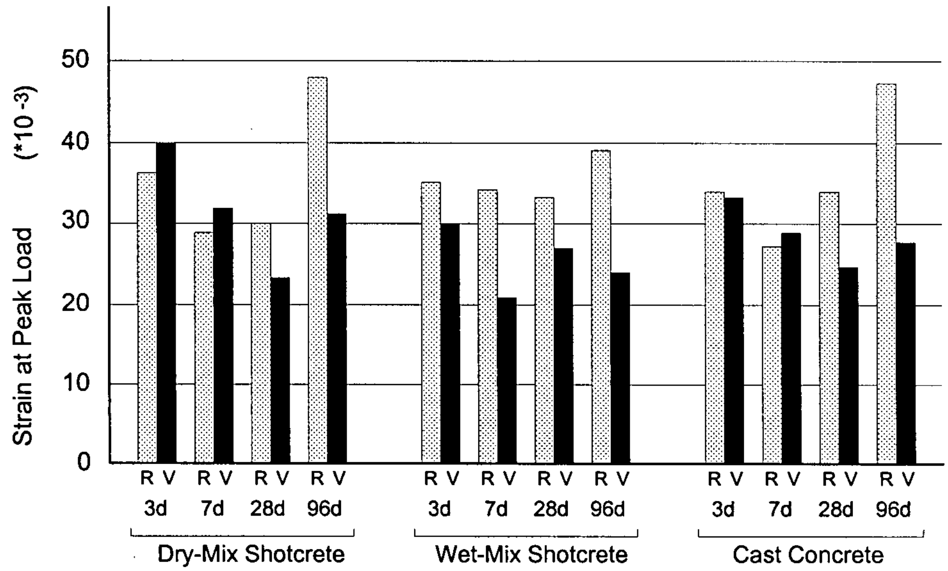


Figure 6.19 Ultimate strain for the various mixes investigated.

perspective of an increasing nonlinearity in the stress-strain curve. Here, it can be seen as a definite increase in the material deformational state. The more widespread distribution of cracks inside the material, many of which can propagate relatively easy, contributes to additional deformations which would not occur in virgin aggregate samples. At the same time, the slippage of unbonded or poorly bonded regions within the specimen under low applied loads promotes even more deformations. The system could be pictured as one that undergoes very rapid deterioration and disintegration; and, as a result of the crumbling of the material, larger deformations occur.

Another possible reason for the larger ultimate strain values observed with recycled aggregate systems is the larger creep in this type of material. At a given load level, the mix with recycled aggregates will allow larger deformations over a mix with virgin aggregates. Hence, over the entire range of displacement from the unloaded point to the peak load, higher deformations would have been incurred in systems with recycled aggregates.

As a final remark, it should be mentioned that materials involving large deformations, as reflected by a larger than normal ultimate strain, are not necessarily unsuitable in practice. There are a number of applications, particularly in shotcrete, where the deformability of the material is more important than the strength itself. For example, in temporary ground support, long term strength or durability may not be of much concern. However, a deformation-allowing system can accommodate rock movements without overstressing the material. In this respect, recycled aggregate shotcrete may be a more promising material for some applications.

6.3.2 Effects of Age and Production Process

From the previous section, it was seen that the extent of cracking and deterioration in the cementitious system would, in a substantial way, determine the amount of strain at peak load. With this in mind, this parameter could serve as an indicator of the accumulated material damage in a system and be used to track changes in the internal structure. One such change is that which takes place as the material ages. To illustrate this, the ultimate strain for the various mixes investigated is plotted as a function of age, as shown in Figure 6.20. As seen, there is no specific trend among the different production processes for a given aggregate type. Rather, the most significant trend that takes place is that between the two aggregate types. The distinction between these two patterns portrayed by the different aggregates will thus be the focus here.

At very early ages, both, recycled and virgin, aggregate mixes start out with similar ultimate strain values. This could be due to the fact that, while recycled aggregate systems are softer with more flaws, the virgin aggregate systems are also still very soft at such young ages. As hydration proceeds and the system ages, it gains a more solid internal structure through the development of a finer microstructure and better bonding between matrix and aggregates. The result is that the system becomes more coherent with less microcracking under applied loads. This constructive evolution in internal

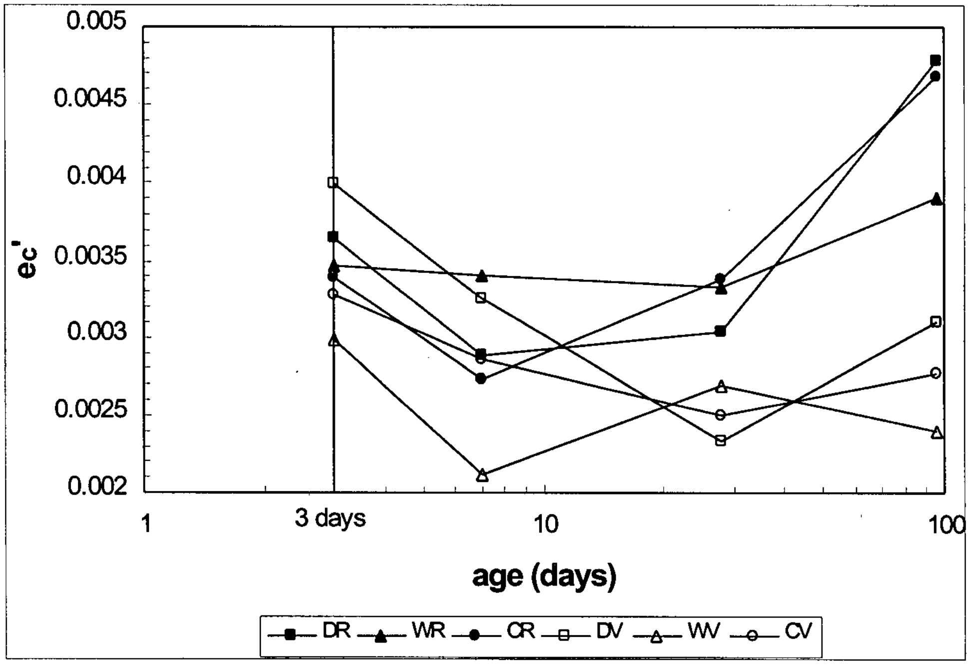


Figure 6.20 Ultimate strain vs. age relationship for different mixes (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

structure remains true in the case of virgin aggregates mixes, leading to the expected gradual decrease in ultimate strain values to almost asymptotic ones, as seen from Figure 6.20. However, in the case of mixes containing recycled aggregates, the ultimate strain seems to be sustained within a narrow band of values over the first 28 days after which it increases by the time the material is 96 days old. This trend is definitely contrary to that portrayed by the virgin aggregate mixes. To explain this sustainment of the ultimate strain over the first 28 days, one has to look at the reasons that impede the constructive development of the microstructure seen with virgin aggregates. The most plausible cause for this is, again, the presence of dust in the system. This fine material will interfere with the formation of a strong, homogeneous, and coherent matrix, as well as with the formation of a well-developed matrix-aggregate bond. As a result, there is a limit as to how far the transition zone can be fully developed. This effect appears to develop at very early ages. Now, to explain the sudden jump in the ultimate strain when the material reaches 96 days of age, recall from Section 6.2.4, there were two reasons presented to explain why recycled aggregate mixes do not follow the same strength gain trend as virgin aggregate mixes: (1) dust in the material, and (2) the potential for adverse chemical reactions to occur at later ages. The first reason would not explain very well the sudden jump, because the presence of dust only limits the degree of material performance enhancement. It does, however, seem possible that deleterious chemical reactions take place in the system and that these create additional flaws from which extra cracking initiates. The reactions that could be induced by the existence of gypsum, glass or brick contamination in the system all have the capability of weakening the internal microstructure rendering it softer and more ductile. Many of these reactions may not necessarily occur at early ages but may continue for an extended period of time. In any case, from the results, it is obvious that these adverse reactions are not negligible since the ultimate strains over a 3 month period increase by over 35-40%.

The above discussion suggests that such deleterious reactions involving certain contaminants do in fact exist but may be manisfested only at later ages. Knowing the properties and behaviour of the material at 28 days is not enough because concrete may portray trends which are opposite from those expected from looking at virgin aggregate mixes. Another implication from this is the importance of investigating the durability of mixes containing recycled material. So far, compressive strength and the ultimate strain at late ages have turned out to be different than expected from virgin aggregate mixes. But, again, in certain applications, these may turn out not to be critical at all.

As far as the production process is concerned, there does not appear to be any specific or unique trend among the different processes employed. What is seen in the virgin aggregate mixes does not reproduce in the recycled aggregate mixes. It is very obvious that the effect of the aggregate type (recycled or virgin) has a far more important influence on the ultimate strain than the effect of the production process (dry-mix shotcrete, wet-mix shotcrete or cast concrete). This observation is also in agreement with the earlier explanations from Section 6.2.3 where it was mentioned that none of the specific processes used produces a smaller percentage reduction in compressive strength. Similarly, here, none of the specific processes used produces a lower ultimate strain. This leads again to the conclusion that the mechanisms by which recycled aggregates deteriorate a cementitious system are similar regardless of the production process employed.

6.4 Fracture Energy

6.4.1 General

As a concrete specimen is loaded, an assortment of internal mechanisms take place inside it, leading to the overloading of the material, its internal deterioration and its eventual failure. These mechanisms include elastic loading, crack initiation and propagation, and slippage of poorly bonded regions, and each of these necessitates a certain amount of energy in order for it to occur. The accumulation of all the energies absorbed by the whole array of internal mechanisms is reflected by the area under the load-deformation diagram characteristic of that particular mode of failure. In other words, the area under such curve represents the amount of work needed to bring about fracture of a specimen. For failure under compression, the compressive stress-strain curves such as those shown in Section 6.1 are used to determine the energy absorbing capacities of the material. This energy also provides an indication of the ductility of the material, with high energy absorptions representing a possibly ductile material.

When the compressive stress-strain tests were carried out, specimens that were very brittle failed in an unstable manner. This behaviour, in addition to the fact that the tests were carried out under open-loop conditions, produced stress-strain curves that would look like that shown in Figure 6.21. Such a curve indicates that large amounts of energy were absorbed by the specimen as it failed and that, perhaps, the specimen is very ductile and even possibly stable. However, such information is misleading if one considers what actually takes place during failure. As the specimen is loaded, besides the energy that is dissipated by crack growth and slipping, a substantial amount of elastic energy is stored in the specimen and in the testing machine. This energy keeps accumulating as crack formation continues. Up to the peak value in the stress-strain curve, the microcracked concrete still maintains a redundant, stable structure and additional strain capacity exists. But, once the main macrocrack has reached its critical length, it continues to grow even if the applied load is reduced. In theory, such reduction in applied load is manifested as a progressive deterioration leading to a strain softening branch. However, in open-loop test setups where large amounts of energy have been stored, such reduction in applied load causes the sudden release of all this sheltered energy producing the sudden unstable growth of a single crack. Since the event occurs so rapidly, much faster than the rate of data acquisition, the softening appears to occur over a large displacement as shown in the figure. The post-peak energy thus determined would contain a dominantly large portion corresponding to the energy originally stored in the machine. To counteract the misinterpretation caused by this observation, points in the curve which occurred during the unstable failure were shifted back, as shown in Figure 6.22. Although there is not enough information to reconstruct the softening branch, such a modified stress-strain curve was deemed to be close enough to the one produced in a fully closed-loop testing system.

With the method described above, stress-strain curves corresponding to specimens which underwent unstable failures were modified. The resulting curves and data points were then used to determine the toughness and ductility related parameters for the specimens. Two such analysis were carried out:

1. absolute energy values
2. post-peak energy ratios

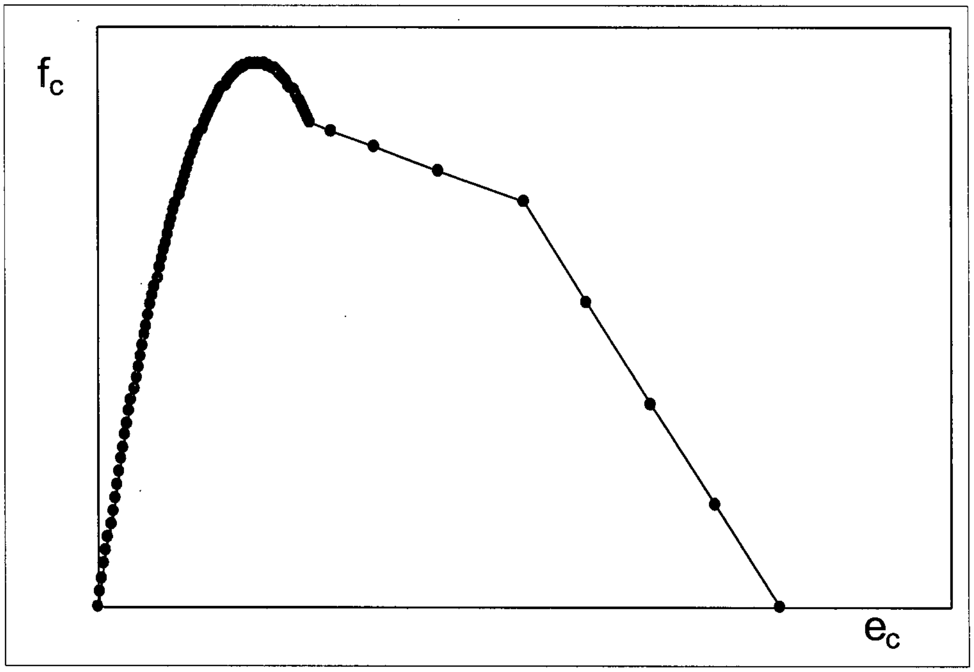


Figure 6.21 Stress-strain curve for a brittle, unstable material.

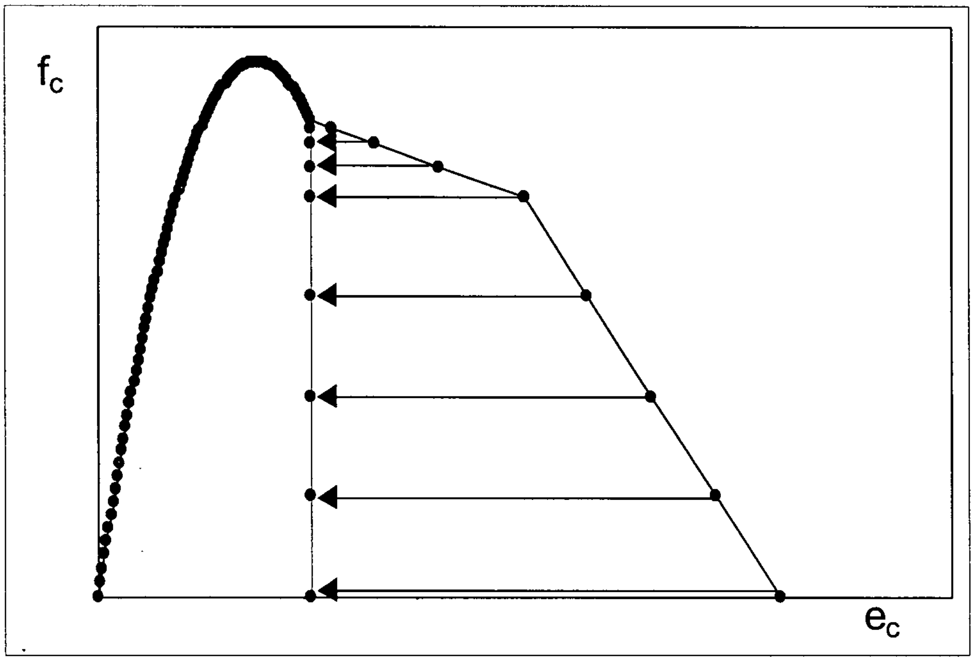


Figure 6.22 Modified stress-strain curve for a brittle, unstable material.

6.4.2 Absolute Energy Values

In this analysis, the toughness values were determined up to a sufficiently large strain value at which it was known that any energy contributions past this point were insignificant. This strain value was chosen to be at 0.045, and the total energy absorbed up to this point was determined from the stress-strain diagram, as shown in Figure 6.23. The results of this analysis are presented in Figure 6.24.

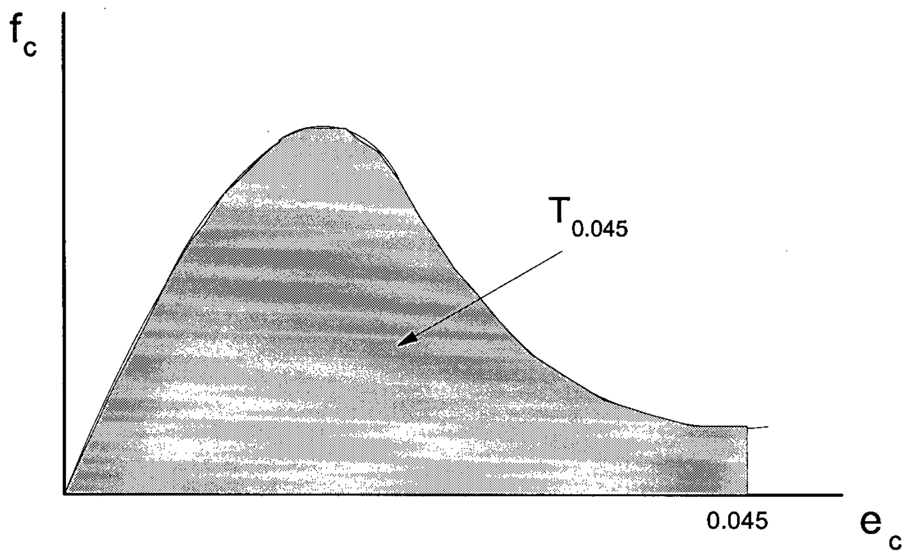


Figure 6.23 Determination of the absolute energy value.

It appears that, other than at early ages (3 days), the mixes with recycled aggregates absorbed larger energies than the mixes with virgin aggregates. This is most likely due to the greater amount of microcracking and internal deterioration that occurs in such mixes. These systems with extensive cracking and deteriorating processes consume more energy than mixes with limited cracking and with deterioration localized to only a few zones. This should come as no surprise considering that mixes with recycled aggregates have a larger number of internal flaws which are widely distributed and can easily propagate.

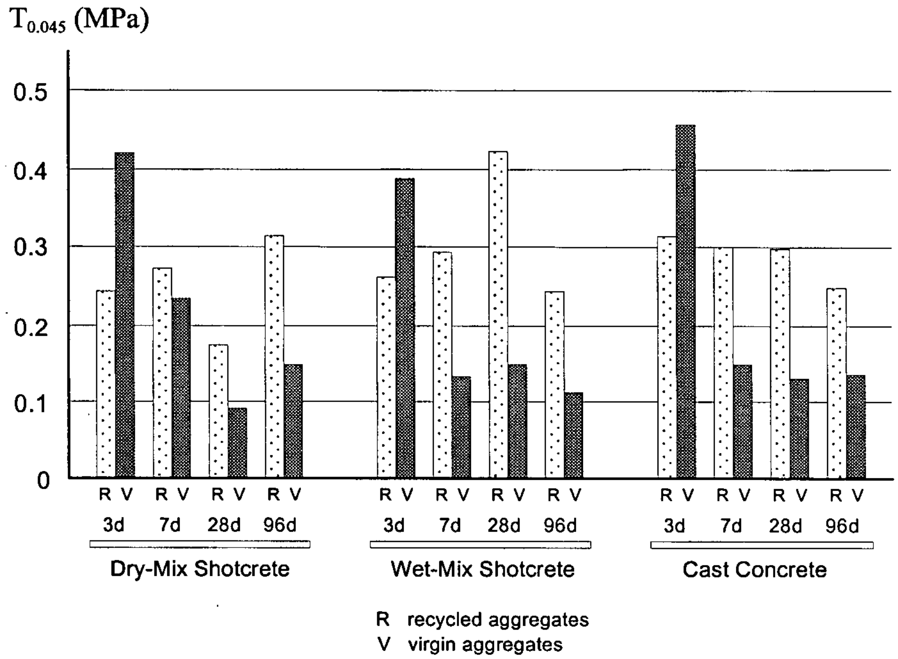


Figure 6.24 Energies absorbed up to 0.045 strain for the mixes investigated.

The above observations on the toughness values seem, though, to depend on the age in question. So, it is helpful to illustrate the age variation of this parameter to understand the behaviour better. Figure 6.25 shows this. From this figure, it can be more clearly seen that virgin aggregate mixes have a high early age absolute energy absorption capacity which decreases with time. On the other hand, recycled aggregate mixes start at some level of absolute toughness and retain it at all ages except for some variations at 28 days. It is interesting to see also that, at very early ages, virgin aggregate mixes actually absorb higher energy than recycled aggregate mixes. However, as the specimens age, the energy absorption capacity of the virgin aggregate mixes drops sharply and remains nearly constant past 28 days. This behaviour can be best explained by considering the energy absorption at two different portions of the curve: prior to peak load and beyond peak load. From the stress-strain curves presented in Section 6.1.1 (Figures 6.1 to 6.12), it could be seen that, after 7 days, all the virgin aggregate mixes had unstable failures, as represented by the sudden sharp drop in the curve. Prior to peak load, energy is mainly absorbed by elastic deformations and by the initiation and growth of cracks, although not extensive ones. But, after the peak load, not much specimen deformation occurs before

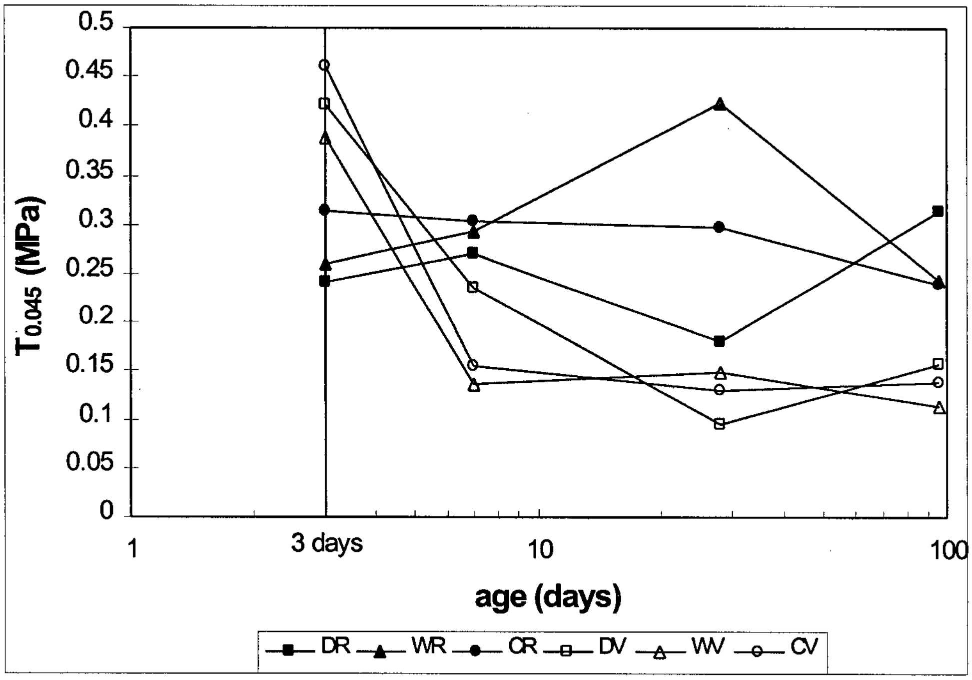


Figure 6.25 Absolute energies vs. age relationship for the mixes examined (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

the critical crack suddenly cuts through the specimen, allowing no room for additional energy absorption. As a result, most of the toughness in the specimen is acquired from the pre-peak mechanisms with very little toughening capacity in the post-peak branch. Thus, the absolute energy values will remain at about the same level past a certain age which, in this case, is 7 days. The situation is different for systems containing recycled aggregates. In the recycled aggregate mixes, the material is soft from the beginning and remains damaged and relatively soft at all times. Prior to peak load, there are some elastic deformations and crack propagation; but, after the peak point, the stable failure and soft bands inside the specimen allow for a gradual decrease in load with room for continued energy absorbing capacity. The result is that both, the pre-peak and the post-peak, portions of the stress-strain diagram contribute to energy intake in the system, as opposed to only pre-peak contributions as is the case with virgin aggregate systems.

Thus, starting from about 7 days, recycled aggregate mixes show a higher absolute toughness value than virgin aggregate mixes. This is exactly what is observed from Figure 6.25.

These points can be made clearer by looking at the pre-peak energy absorption values. Figure 6.26 shows the variation of the energy absorbed up to peak load with age, for all the mixes considered. It can be seen that virgin aggregate mixes have a toughness to peak load that does not vary much with time, and this is what remains dominant when considering absolute energy values, as the post-peak contribution is very limited. On the other hand, recycled aggregate mixes have an increasing toughness to peak load which gets added to a supposedly increasing post-peak contribution resulting in much higher total absorption numbers. Now, the only reasons to explain why virgin aggregate mixes

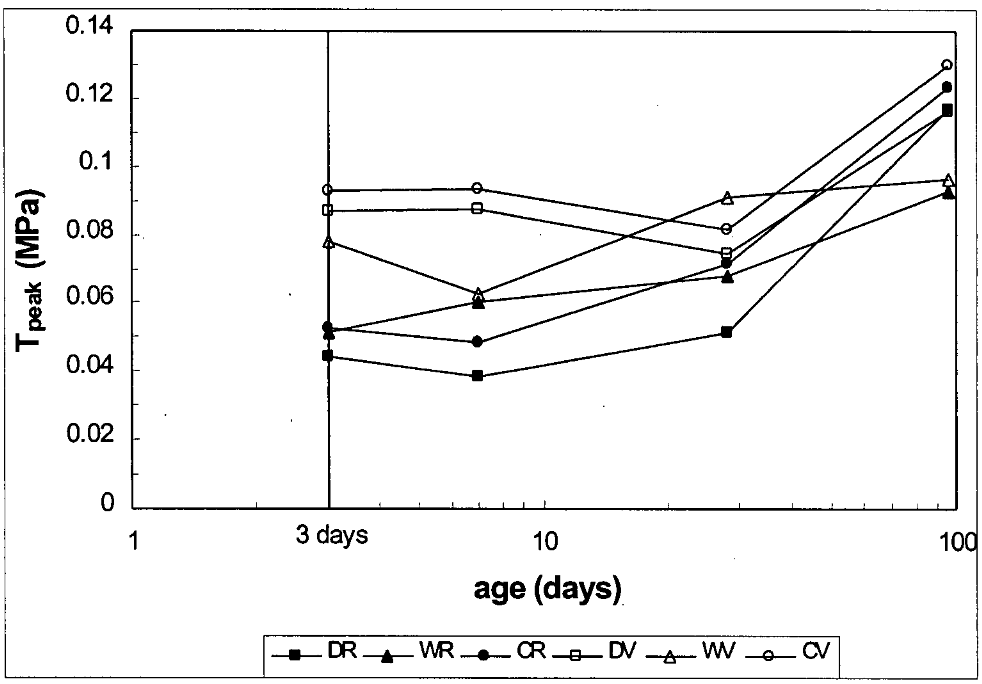


Figure 6.26 Energy up to peak load vs. age relationship for the mixes examined (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

show higher absolute toughness values at 3 days than recycled aggregate ones are: the material is still soft and can sustain stable failure, and the peak loads are higher, thus absorbing more energy.

The above considerations suggest that recycled aggregate mixes are systems which are, not only stable, but also, more ductile, and overall, more energy absorbing. In virgin aggregate mixes, the pre-peak energy absorption capacity appears to be mainly due to the elastic deformations as seen from the nearly linear stress-strain diagrams, particularly at late ages. However, with recycled aggregates, the systems tend to undergo some degree of elastic deformation, but more significantly, the more extensive cracking provides for the larger energy absorbing capability. This absorbed energy also provides the advantage of dissipating the stored elastic energy from the system, either from the specimen or from the machine, resulting in a stable decrease of the applied load past the peak load and, consequently, a stable softening branch. As a result, in such systems, the post-peak energy absorbing capacity becomes a useful parameter in determining the kind of stability existing in the system, as well as the amount of additional ductility that the system has remaining beyond peak load. This aspect will be considered in more detail in the next section.

As far as the production process is concerned, no particular trends are evident. This suggests again that the effect of the recycled aggregates is more important than that of the process itself.

6.4.3 Post-Peak Energy Ratios

In this analysis, the post-peak toughness values up to specified strain values were taken as a proportion of the pre-peak toughness value, as an energy ratio. The strain values were chosen to be at 0.01, 0.02 and 0.03. This method is illustrated in Figure 6.27. These ratios provide an indication of the level of post-peak ductility that the material has. A ratio of unity means that, past the peak load, there is still as much energy absorbing capacity as there is prior to the peak load, regardless of the failure mechanisms. A low ratio means that, beyond peak load, the material cannot sustain further loads over additional displacements. Another implication from this analysis is regarding the stability of the system. When a system is not stable, it will lose its load carrying capacity very rapidly right after peak load with no additional energy absorption capability. So, a low ratio would also be indicative of a possibly unstable system.

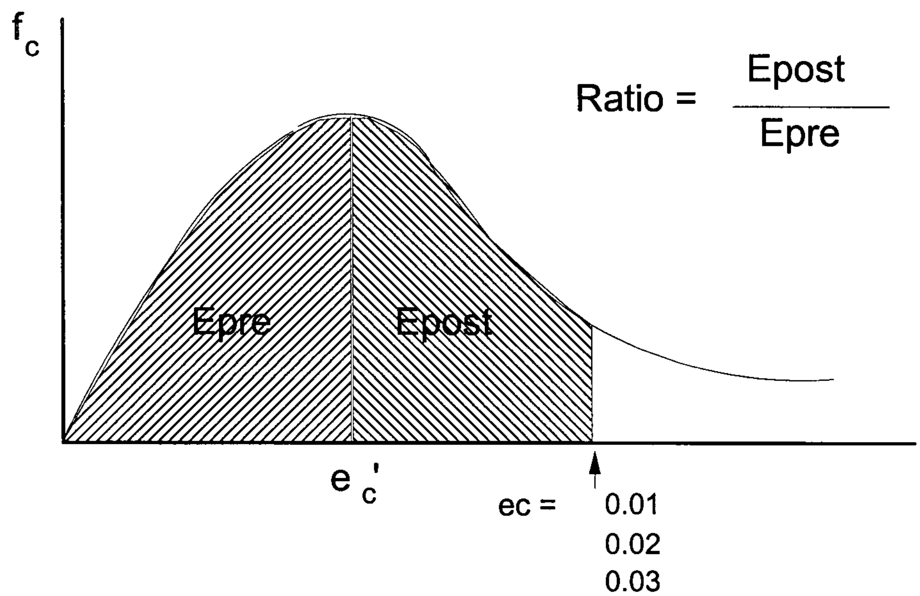


Figure 6.27 Determination of the post-peak energy ratios.

The results of this analysis are shown in Figure 6.28. Note that mixes containing recycled aggregates display much higher post-peak energy absorbing ratios compared to virgin aggregate mixes. In some instances, these ratios reach values up to 2 and 3 at a strain of 0.01 and values up to 5 at a strain of 0.03. An interesting observation, however,

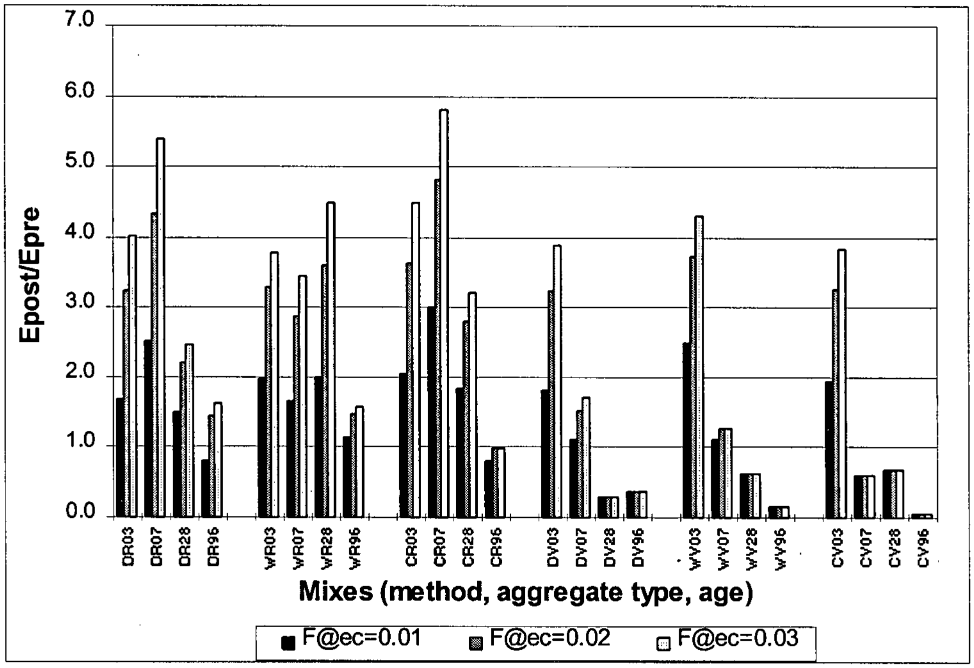


Figure 6.28 Post-peak energy ratios for the mixes investigated (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

is the fact that specimen age does not seem to affect these ratios much in the recycled aggregate mixes. Except for a reduction at 96 days, the ratios are generally above 2 at other ages, even at a strain of 0.01. This indicates that recycled aggregate systems can have or can sustain energy intakes that are even twice as much as those sustained in the pre-peak regime. Even at 96 days, there is still as much post-peak energy absorption as there is prior to peak. On the other hand, virgin aggregate mixes display ratios at 3 days that are comparable to those of recycled aggregates, but these ratios quickly diminish with age. Past 28 days, the ratios are down to less than unity, and at 96 days, almost no post-peak ductility exists in the system. The effect of material strengthening with age can be seen in these virgin aggregate systems. As hydration proceeds, the material continuously gains a finer microstructure and stronger interfacial bonds which altogether, not only makes the system stronger, but also renders it more brittle. Whereas, in recycled aggregate systems, there is always a substantial amount of defects in the system, whether as initial flaws or as ones that develop with time, to allow for the stable gradual deterioration of the structure which, in the process of so doing, consumes most of the applied energy.

The preceding observations suggest that systems with recycled aggregates do, in fact, possess a significant potential for absorbing energy even after the peak stress has been reached. It can be postulated that this behaviour is caused by the large number of distributed flaws in the system, which lead to the slow and gradual degradation of the matrix as well as the interfacial zones. As a result of the substantial amounts of energy being dissipated by such mechanisms before peak load, there is not enough elastic energy stored either in the specimen or in the machine to produce a catastrophic failure of the critical crack at peak load. Consequently, past the peak load, this crack is still relatively intact and has a large capacity to dissipate more energy through a number of other processes, as it propagates through the material. These processes are different from those occurring in the pre-peak stage, and they include frictional sliding, shearing of soft material regions, breakage of soft contaminants and debonding of aggregates, as shown in Figure 6.29. Such mechanisms are not all present in virgin aggregate systems which is why they do not portray any significant post-peak ductility. Evidence of these processes in recycled aggregate mixes was seen from the fractured specimens. A visual examination of the broken pieces revealed a large amount of aggregate pullout, accompanied by shearing through flexible material like asphalt and wood, all surrounded in a soft matrix containing pockets of dust and powder. On the other hand, virgin aggregate specimens broke along single clean surfaces which encompassed a large amount of fracture through aggregates. From these observations, it can be concluded that the tortuosity of the crack growth path in recycled aggregate mixes, along with the number of plastic regions this crack has to grow through, are what provides the ample amounts of post-peak energy absorption capacity in these systems; and, these would be existent regardless of the age of the material or the process by which the specimens were prepared.

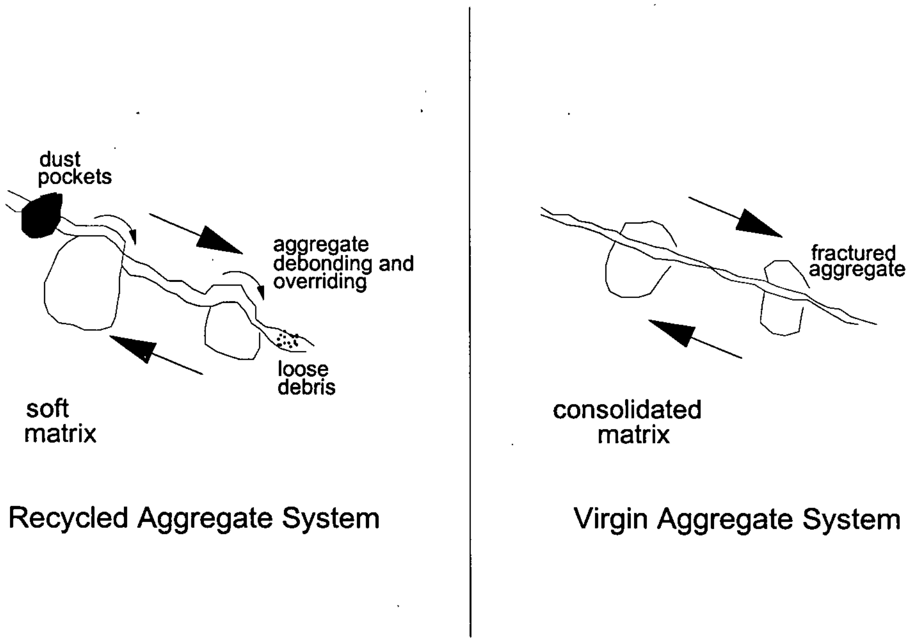


Figure 6.29 Post-peak energy absorbing mechanisms.

These results suggest that recycled aggregate systems may find promise in applications where large amounts of material ductility are required. While its strengths are not very high, the greater deformability and energy absorbing capacity can be valuable, particularly in applications where these attributes are necessary, such as intemporary ground support. At the same time, there is less concern with the peak stress of the material because, as seen, even past the peak stress point, considerable amounts of energy can still be absorbed.

6.5 Modulus of Elasticity

6.5.1 Test Results and Discussion

Concrete is not a truly elastic material. The nonlinearities of the stress-strain curve are due to an irreversible response of the concrete which can actually be measured when the concrete is loaded and then unloaded. Thus, there is difficulty in determining the modulus of elasticity from such a nonlinear diagram. The practical method to do this, as stipulated by the ASTM C469, is to take the chord modulus between two points in the curve. The modulus of elasticity is normally used in design to predict the response of the material assuming an elastic response from it which is fairly reasonable at small strain values.

As soon as concrete is loaded, microcrack initiation and propagation takes place within the system. The formation of these cracks gives concrete its inelastic behaviour as the damage becomes permanent. At the same time, these cracks degrade the system rendering it softer and weaker, thus causing the gradually increasing nonlinearity seen in the stress-strain curves. So, the chord modulus of elasticity includes an element of nonlinearity and its value would therefore depend on the two points at which it is taken. However, for the two points set by the ASTM C469, the material is still relatively linear and hence the use of an elastic modulus is still applicable; but, it is clearly seen that the degree of internal cracking and material damage will have a major influence on this material parameter.

Results for the chord modulus of elasticity determined in this manner from the stress-strain curves of Figures 6.1 to 6.12 are shown in Figure 6.30. The values for the virgin aggregate mixes lie in the range normally expected for virgin aggregate concrete and virgin aggregate shotcrete, which is about 20-40 GPa. However, it can be seen that the modulus of elasticity (E) is much lower for recycled aggregate mixes. This is the same observation that has been made by other researchers as well. The lower modulus of elasticity of recycled aggregate mixes implies again that this material is softer and more flexible, although this flexibility is not completely recoverable.

Looking at the above values is useful to get an idea of the E values that can be obtained from the mixes; but, again, the useful parameter to consider here is the percent reduction in this property when recycled aggregates are used in place of the virgin aggregates.

These values are shown in Table 6.2 for all the different mixes investigated. The results indicate that substantial reductions in the E value are possible when crushed waste concrete is used as aggregate replacements. Such reductions are slightly above those reported in the literature which are typically in the range of 20-40%. Again, variations in the aggregate batches and the variability of results caused by the presence of copious amounts of fine material are likely the reasons for the observed reduction in the E value.

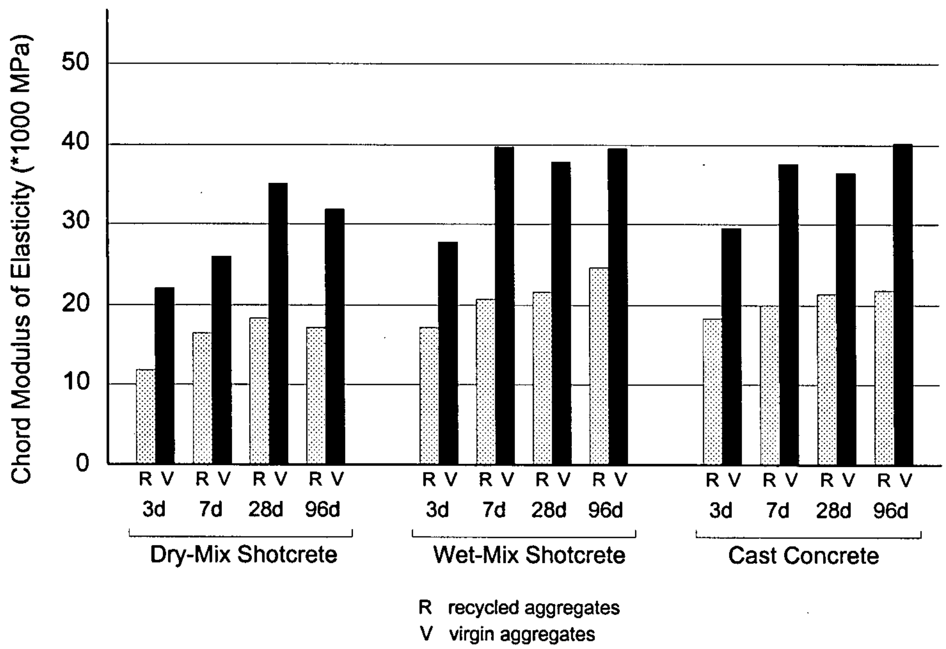


Figure 6.30 Modulus of elasticity for the mixes investigated.

Table 6.2 Percentage reduction in modulus of elasticity for mixes with recycled aggregates.

Age	Percentage Reduction (%)		
	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete
3 days	47	38	38
7 days	36	48	47
28 days	48	44	42
96 days	46	39	47

To understand the causes for the reductions in the modulus of elasticity, it is helpful to consider the processes that contribute to this property. The elastic modulus of a composite depends on the modulus of the component phases and the strength of the bonds that exist between the different phases. As far as the matrix qualities are concerned, its porosity and water-cement ratio have great influences over the modulus of elasticity. It is then easy to deduce from this that the matrix in recycled aggregate mixes has a partial contribution to the reduction of the material property in question. The high content of fine material and dust pockets, as well as poorly cohered regions, makes the matrix significantly softer than in mixes made with virgin aggregates. The other factor that influences the modulus of elasticity of concrete is the corresponding modulus of the aggregates. Crushed waste concrete is known to have a generally lower elastic modulus than normal aggregates because of the softer attached old matrix and the softer contaminants coexisting in the waste stream. As a result of their lower stiffnesses, the overall composite stiffness is also reduced. The last and probably most important reason for the lower elastic modulus of recycled aggregate mixes is the nature of aggregate-matrix bond. Only with a strong enough bond can the full stiffnesses of the individual phases be realized to promote the overall composite stiffness. However, with a bond that degrades and loses its load transferring capacity, the composite cannot fully achieve its potential modulus of elasticity. Such is the case in recycled aggregate mixes as it is already known that the interfacial regions contain severe defects which quickly multiply and cause premature debonding between the matrix and the aggregates. As a result of the inferior quality of these three components: matrix, aggregates and bond, it should come as no surprise that the elastic modulus of recycled aggregate systems is invariably lower than that of virgin aggregate mixes. The other question is the degree to which recycled aggregate mixes are inferior to their virgin aggregate counterparts. This has already been determined to be in the range of 36-48%.

Another interesting observation is the extent to which the deteriorating mechanisms affect the modulus of elasticity, compared to the extent to which these mechanisms affect compressive strength. The elastic modulus and compressive strength of concrete are linked by a relationship of the following form: $E \alpha (f_c')^{1/2}$. This indicates that the modulus of elasticity should not be affected as much as the compressive strength is. This is exactly what the results indicate by noting that the percentage reduction in the modulus of elasticity, as mentioned above, lies in the range of 36-48%, whereas the percentage reduction in compressive strength lies in the range of 40-56%. Such observations imply that, although the mechanisms by which the elastic modulus and the compressive strength of concrete are affected may be similar in nature, the influences on each of these properties are not manifested in the same proportions. Another factor that could explain the lesser percentage reduction in the modulus of elasticity compared to the compressive strength is the fact that the modulus of elasticity is only determined at up to 40% of the peak load, and the material, at such load levels, still does not exhibit much of the detrimental effects on the stress-strain curve that are caused by a damaged internal structure.

As far as age effects are concerned, the variation of the modulus of elasticity with specimen age is plotted in Figure 6.31. These results show that, for all mixes, the values for the modulus of elasticity reach a certain level at early ages but remain fairly constant for the time period investigated. This behaviour is similar to that seen with compressive strength results and can be related to the fact that these two material properties are linked, thus displaying similar trends. In the case of compressive strength, it was discussed that the probable reasons for a slowed down strength gain for recycled aggregate mixes were the presence of dusted regions which impede continued strengthening of the structure and the continuation of harmful reactions over the long term which would deteriorate the internal structure with time. These factors are likely to be the same ones limiting the continued increase in the elastic modulus of such mixes.

As far as the production process is concerned, once again there does not appear to be any influence of the process on the results obtained. Dry-mix shotcrete with either, recycled or virgin, aggregates show lower modulus of elasticity values than obtained by the other two production processes. However, this is most likely related to the lower compressive strengths that such mixes exhibit. The lack of difference in results between the different processes suggests again the fact that recycled aggregates deteriorate cementitious systems in the same manner, regardless of which production method was employed, and none of these methods proves to be more beneficial than the other.

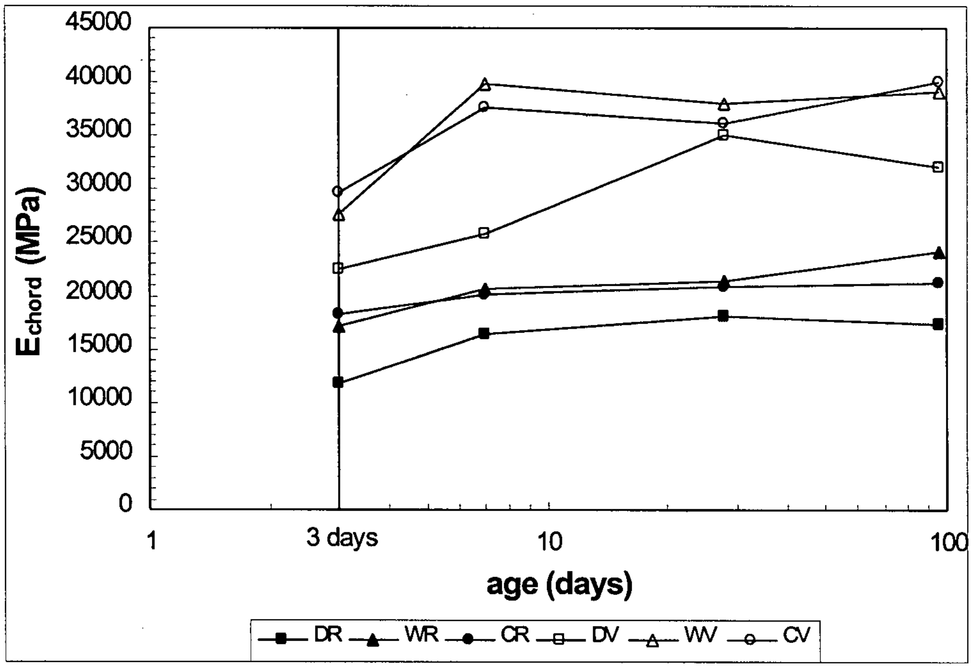


Figure 6.31 Modulus of elasticity vs. age relationship for the mixes investigated (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

6.5.2 Damage Quantification

From the previous section, it was discussed how the modulus of elasticity of concrete can be affected by the internal structure and by the changes that occur in it. A strong matrix attached to sound aggregates through a strong bonds will result in higher modulus of elasticity values. However, the deterioration of any of these components or a combination of these components will lead to the reduction of this material property.

Such deterioration is normally dominated by the growth of microcracks and their coalition which renders the overall system more deformable while not taking much additional load. As mentioned before, the chord modulus contains an element of nonlinearity from the stress-strain diagram due to the way in which it is measured. As the material progressively deteriorates and becomes more dominated by extensions of these cracks, the nonlinearity in the stress-strain diagram becomes more conspicuous with an increasing load. Thus, the modulus of elasticity measured at increasing strain levels would be progressively lower as a consequence of the larger amount of accumulated internal damage in the system. This behaviour is known as stiffness degradation; and systems that have been severely damaged prior to any load application will degrade very rapidly when they are stressed.

Back in Section 6.3, it was mentioned how the strain at peak load (ultimate strain) could provide an indication of the damage condition in the system. Since this damage is also the cause of degradation in the stiffness of the material, the modulus of elasticity taken at different parts of the stress-strain curve can then be used to represent the amount of this accumulated damage, as well. This is done so by defining a damage parameter equal to the ratio of the chord modulus up to 40% of the peak load to the chord modulus up to the peak load, as shown in Figure 6.32. It can be seen that a very well-formed material which behaves close to linear elastic and with very little damage has an almost linear stress-strain response up to peak load thus, yielding a value of the damage parameter close to unity. On the other hand, a very soft material which degrades very easily under the application of loads exhibits large strains and significant nonlinearities in the stress-strain curve thus, yielding values of this parameter much higher than unity. Larger values of this damage parameter are indicative of a greater extent of accumulated damage in the system.

The results obtained for this damage parameter for the different mixes investigated are plotted as a function of age in Figure 6.33. This figure shows more clearly the changes that take place inside the systems as they age. Generally, it can be seen that, at all ages,

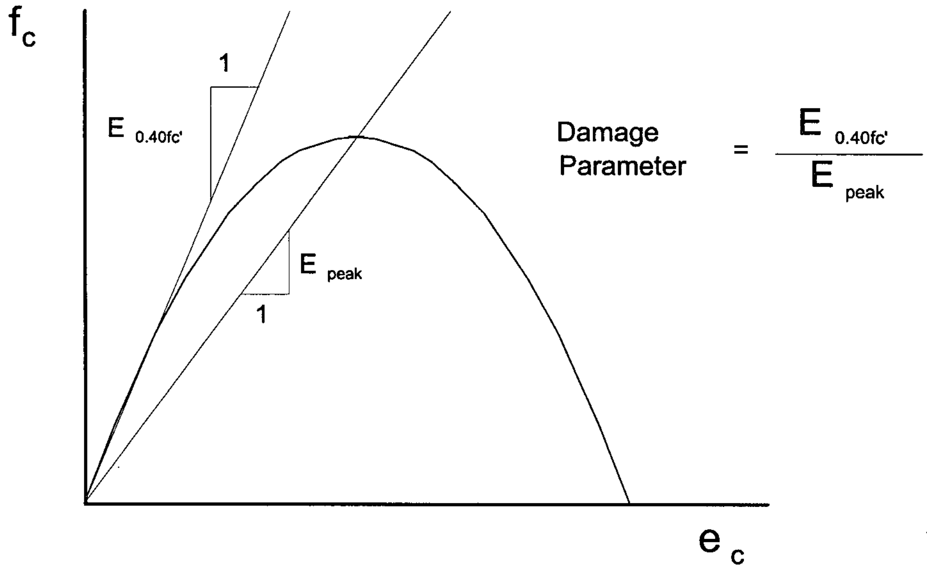


Figure 6.32 Definition of a damage parameter.

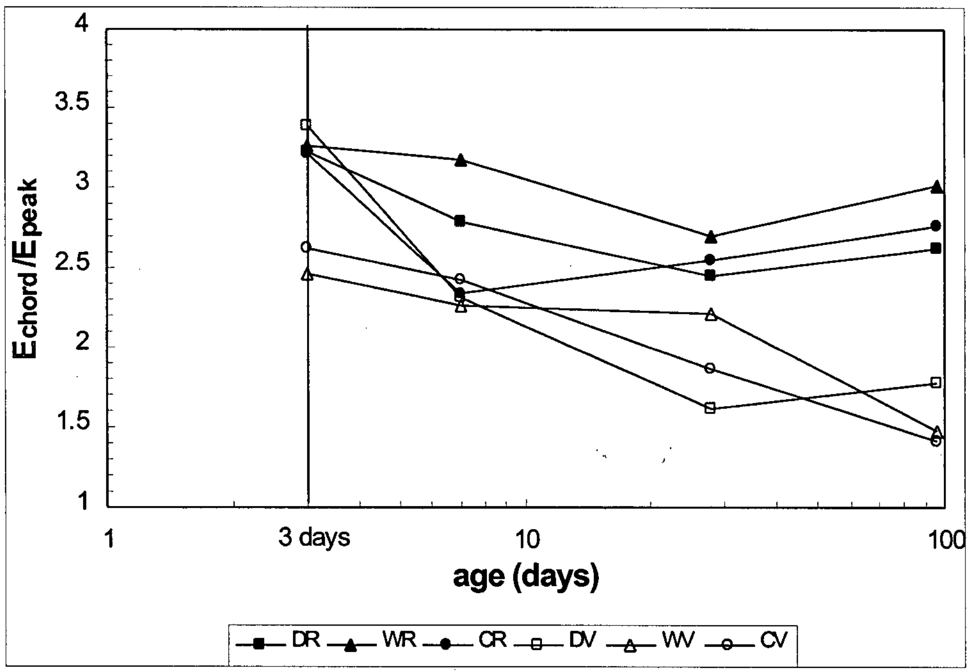


Figure 6.33 Damage parameter vs. age relationship for the different mixes examined (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

recycled aggregate mixes have higher values of the damage parameter than virgin aggregate mixes, indicating that the former systems undergo more degradation and are subject to more internal damage when they are loaded, no matter how old the samples are. Considering age effects, it can be seen that, as the specimens age, virgin aggregate mixes show a large and a continuous decrease in the chosen damage parameter. While recycled aggregate mixes show some decrease in this parameter, it is very limited. Even at early ages, virgin aggregate mixes have lower values of the parameter compared to recycled aggregate mixes implying that such mixes, although still being soft at young ages, are less prone to the extensive damage that is seen in recycled aggregate mixes. Virgin aggregate mixes, as they age, continue to solidify their structure and their bonds. Hence, cracking and damage in the structure becomes very limited and localized making the system very stiff even at strain values close to the ultimate strain. The result is an almost linear pre-peak stress-strain response, as seen in Figures 6.1 through 6.12, and a value of the damage parameter somewhere around 2. On the contrary, recycled aggregate mixes contain a large amount of pre-existing flaws that limit the extent to which strengthening can occur. So, these systems already have a degree of defectiveness in them to start with and the damage values do not decrease with time but rather, remain within a small range at all ages of test. An equally important observation regarding mixes with recycled aggregates is that, at late ages (96 days), the mixes even display some slight increase of the parameter, indicating that additional damage has taken place in the structure even prior to load application. This again emphasizes the hypothesis that there may be some internal mechanisms, such as a deleterious chemical reactions, which induce extra damage to the system and which occur over extended periods of time. This must be the case because, if such mechanisms did not exist, the damage parameter would remain at about the same values at all ages. As far as the production process is concerned, there does not appear to be any trend portrayed by any of the processes, thus indicating that damage occurs in the same way in all systems regardless of the method in which the material was produced.

The use of a damage parameter such as that proposed here is very useful as an alternate proof to verify the presence of mechanisms that were hypothesized from observations of other results discussed in the previous sections. It has once again been demonstrated that the extent to which recycled aggregate mixes gain a coherent structure is limited no matter how long the specimens age; and, most importantly, that injurious mechanisms exist that take place with time rendering the system weaker and more vulnerable to extensive damage. So, while virgin aggregate systems grow stronger and stiffer, recycled aggregate mixes become softer and more flexible. The use of this damage parameter to analyze mixes at much longer ages helps provide an idea of what the strength and other material properties could turn out to be at those ages. And, finally, by looking at the situation from the damage variable point of view, it is seen that none of the production processes studied provides an advantage over the other ones, in terms of a better utilization of recycled aggregate.

6.5.3 Modulus of Elasticity vs. Compressive Strength Relationship

The modulus of elasticity used in concrete design is seldom determined by direct test but is generally estimated from an empirical relationship between modulus and compressive strength. This relationship has a well established form of the following type:

$$E = K\,(f_c')^{1/2}$$

The form of this relationship was evident in the results obtained from the mixes investigated by noting that the modulus of elasticity is not affected to same extent as the compressive strength when recycled aggregates are used. In the ACI Building Code [73], K for normal density concrete has a value of 57,000 if E and f_c' are in psi units, or a value of 4,730 if E and f_c' are in MPa units. Although this expression may reasonably estimate the elastic modulus of virgin aggregate mixes, it may not necessarily apply equally to recycled aggregate mixes. This had to be determined by finding the corresponding coefficient for recycled aggregate mixes.

For each specimen tested, its modulus of elasticity, E, and compressive strength, f_c', values were compiled. Linear regression was then performed between E and $(f_c')^{1/2}$, once on all virgin aggregate samples and once again on all recycled aggregate samples. The resulting coefficient for the virgin aggregate samples would verify the accuracy of the coefficient given by ACI 318, while the coefficient for the recycled aggregate samples would indicate if a different value of it is required. Results of the linear regression analysis are shown in Figure 6.34. The variation of elastic modulus with age would have already been taken into account by the variation of the compressive strength with age.

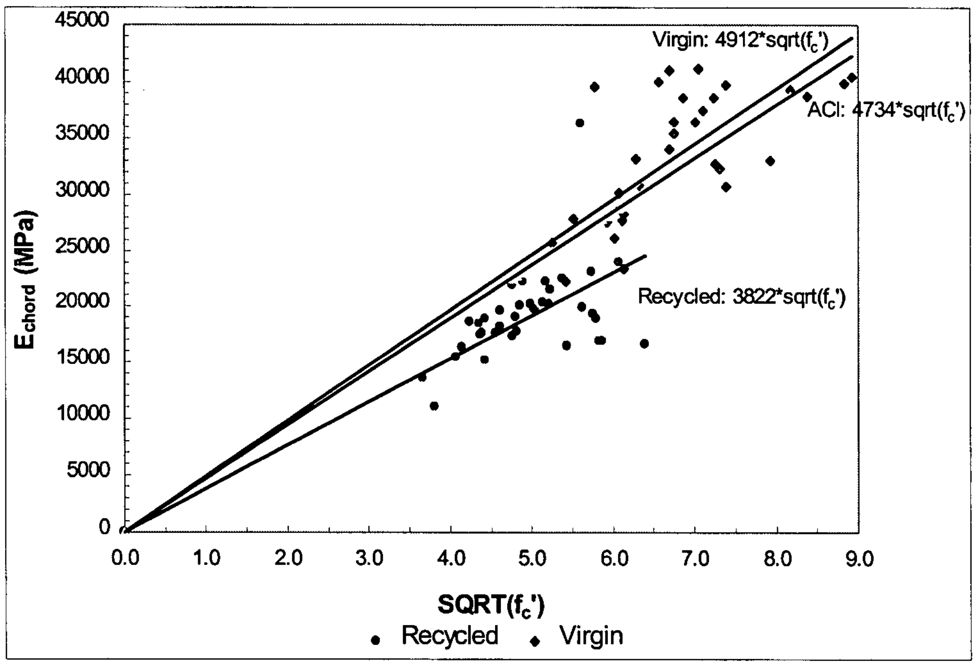


Figure 6.34 E vs. f_c' relationship for recycled and virgin aggregates.

From the plot, it can be seen that the mixes with virgin aggregates approximates that stipulated by ACI 318, thus the results are fairly consistent with what the design code establishes. However, mixes with recycled aggregates do show a definitely lower value of this coefficient at K = 3,822 compared to K = 4,912 obtained with virgin aggregate mixes. So, the lower compressive strength in mixes involving recycled aggregates is not the only factor leading to a lower modulus. There is a possibility that there are other mechanisms or factors in recycled aggregate systems that affect the elastic modulus more than the compressive strength itself. It could be a softer matrix; presence of contaminants; or a rapid debonding between matrix and aggregates. The end result is that, if crushed waste concrete is used in design, attention must be paid to the value of K chosen. It is definitely not the same as that obtained with ordinary concrete.

6.6 Splitting Tensile Strength

6.6.1 General

The splitting tensile strength test is one of the easiest and most commonly used methods to determine the tensile strength of concrete. It is an indirect measure of the tensile strength of concrete since direct tensile strength tests are difficult and not very practical to perform. However, because of the different volumes under maximum stress and the distinct distribution of stresses in the specimen compared to direct tensile test specimens, splitting tensile strengths are normally higher than the actual direct tensile strengths. They are typically higher by about 15% for conventional concretes. As a proportion of the compressive strength, the splitting tensile strength is about 8-14% of it which is slightly higher than the 7-11% obtained from direct tensile tests.

When a concrete specimen is subject to a tensile load, the process of internal deterioration leading to eventual fracture of the entire specimen is similar to that for compression testing. Microcracks form mainly at the interface due to flaws and defects, and start propagating around the aggregates and into the matrix. They eventually interconnect, forming a network which, past a certain value of stress, coalesce into a major macrocrack which becomes the critical one. Peak load is then reached when this major crack reaches its critical length. Softening is also present and it depends on various factors like particle size and type, water-cement ratio, porosity, etc.

One major difference between the growth of cracks in tension as opposed to that in compression is that, in tension, the cracks propagate in a direction perpendicular to the applied load compared to a parallel propagation for specimens under compression. This is due to the cracks being primarily exposed in their weakest failure mode, which is as a Mode I type of failure, when subjected to tensile loads. Whereas, in compression, mixed mode failure occurs and cracks have to grow in different directions, eventually as a major parallel crack. Another significant difference between tensile and compressive failure is that, since failure in tension involves significant matrix-aggregate debonding which controls the process, the tensile strength is more sensitive to aggregate variations than compressive strength. This sensitivity has been noted with other variables such as aggregate size and aggregate texture [45]. It would be interesting to see whether or not recycled aggregates affect the tensile strength of concrete in the same way that they affect compressive strength. The following sections look at the results obtained for the splitting tensile tests on the mixes investigated.

6.6.2 Results and Discussion

The results for the splitting tensile strengths, determined according to the methods of ASTM C496, are shown in Figure 6.35. Not surprisingly, the tensile strengths of recycled aggregate mixes are lower than those of virgin aggregate mixes. Just as the compressive strengths of mixes made with recycled aggregates are lower than the compressive strengths of mixes made with virgin aggregates, it should be expected that the same trend applies to splitting tensile strengths, although the fracturing processes are different in nature. Due to the way in which the material is loaded, cracks are mainly subjected to a tensile mode of opening, making them grow in a preferential direction perpendicular to the applied load. The strength of the material, thus, relies heavily on the cohesiveness of the matrix and on the strength of the interfacial zone to resist debonding and separation under tensile stresses. The main reason for such a decrease in the tensile strength of recycled aggregate mixes is the inferior bonds in this kind of system. If the matrix is very soft and the aggregates can separate easily, cracks can form and grow in a fairly easy and unresisted manner at lower values of the applied load. For example, if an aggregate is coated with dust, the matrix will not bond well and these two can be separated without much resistance. The same can be said about bonding between matrix and other foreign substances. Basically, many of the dusted regions or weak particles provide unresisted paths for crack growth. Poor bonds, in addition to a softer and weaker matrix, are what lead to the lower peak loads. The lower resistance to crack initiation and propagation also leads to the conclusion that recycled aggregate matrices or concretes have lower values of the fracture toughness, K_C. In this case, the fracture toughness parameter in question would be the Mode I type, K_{IC}, since tensile failure is predominant. Compared to the published data that exists in the literature, these results fall within the same range which is 2.4-4.5 MPa (from Section 3.5.5). Thus, the mixes tested give similar tensile strength values to mixes in other investigations, regardless of the mix proportions or type of recycled aggregates used.

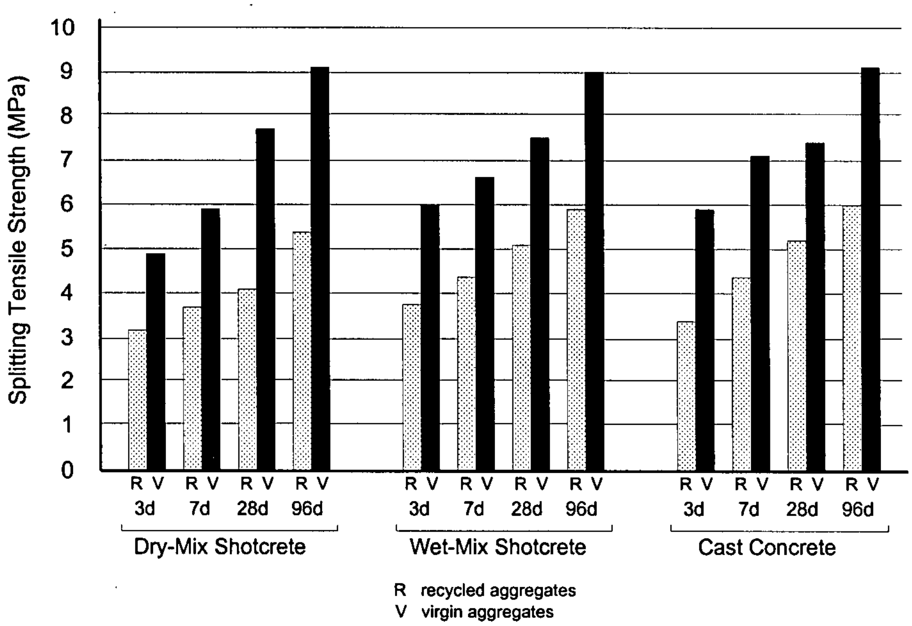


Figure 6.35 Splitting tensile strengths of the mixes investigated.

In terms of the loss of potentially achievable tensile strength, the percentage reduction of this parameter can be looked at. Table 6.3 summarizes these results for the different mixes investigated. These values are larger than the percentage reductions reported in the literature. Published values have ranged up to 20%, but the larger differences here are probably due to differences in the nature of the recycled aggregates. An interesting aspect to note is that the percentage reductions for the splitting tensile strength, as tabulated in Table 6.3 and ranging from 30-47%, are lower than the percentage reductions for the compressive strength, tabulated in Table 6.1 which ranging from 40-56%. This depicts a peculiar point that the tensile strength of concrete or shotcrete is not affected by

Table 6.3 Percentage reduction in splitting tensile strength for mixes with recycled aggregates.

Age	Percentage Reduction (%)		
	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete
3 days	35	37	42
7 days	37	33	38
28 days	47	32	30
96 days	41	34	34

recycled aggregates as much as compressive strength. So, probably, there is some kind of mechanism occurring in compression which does not occur in tension and the detrimental influence of recycled aggregates is manifested in this mechanism. While the tensile strength of concrete depends more on the bonding between different phases, the compressive strength depends more on the matrix characteristics. From these results then, it is possible that the presence of recycled aggregates, in the coarse and in the fine fractions, has a very detrimental influence on the quality of the matrix. It appears that the coarse aggregates particles with some surface attached old mortar and the contaminants do not pose as harmful an effect as the fine dust that gets blended and becomes part of the matrix. This dust probably does not have any further cementing properties anymore nor can it act as a nucleating agent nor is it fine enough to act as a filler; thus, it just becomes a diluting substance which impairs the matrix from developing its potential strength. On top of that, the dust may contain harmful chemicals and organic material in fine form. Altogether, these factors render the matrix very weak and soft. This was observed from the broken samples which showed that the matrix failure surfaces had almost no cohesion at all. Such observations suggest that, from a compressive strength standpoint alone, the inclusion of fine material from the waste concrete should be avoided. Perhaps, lower percentage reductions in compressive strength may be achievable if the fine material is screened off, but this remains to be proven.

A further aspect that could be considered is the ratio of the splitting tensile strength to compressive strength. For the different mixes investigated, this is shown in Table 6.4.

Table 6.4 Ratio of splitting tensile strength to compressive strength for the mixes investigated.

	f_{st}/f_c'					
	Recycled Aggregates			Virgin Aggregates		
Age	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete	Dry-Mix Shotcrete	Wet-Mix Shotcrete	Cast Concrete
3 days	0.22	0.20	0.17	0.18	0.17	0.15
7 days	0.21	0.19	0.18	0.16	0.17	0.16
28 days	0.18	0.19	0.18	0.16	0.16	0.15
96 days	0.17	0.17	0.17	0.16	0.14	0.12

The ratios for the virgin aggregate mixes are closer to those expected. Recall from Section 6.6.1 that this ratio is in the range of 8-14%. However, for recycled aggregates, this ratio is higher. The tensile strength, which is not as sensitive to recycled aggregates, divided by the more sensitive compressive strength yields a higher value of the ratio. This again proves that for the specific type of recycled aggregates used here its detrimental effects on compressive strength are not as significant for the tensile strength. It would be interesting to study this further using concepts of micromechanics.

6.6.3 Effects of Age and Production Process

Like the stress-strain tests, the splitting tensile tests were carried out at 3, 7, 28, and 96 days of age. This would indicate any changes with time that took place within the material. The tensile strengths, plotted as a function of age, are shown in Figure 6.36. It can be seen that both, virgin and recycled, aggregate mixes gain tensile strength at similar rates, slightly faster at early ages and leveling off at later ages. It appears that, unlike compressive strength gain, there is no rapid leveling off of the tensile strength gain for recycled aggregate mixes. One possible implication from these results then is that the deleterious chemical reactions taking place over extended periods of time do not affect the matrix-aggregate bond as much as the matrix. As a result, the tensile strength gain does not get as severely interrupted over time and the material can continue gaining tensile strength in a similar, though slightly lower, rate compared to virgin aggregate mixes.

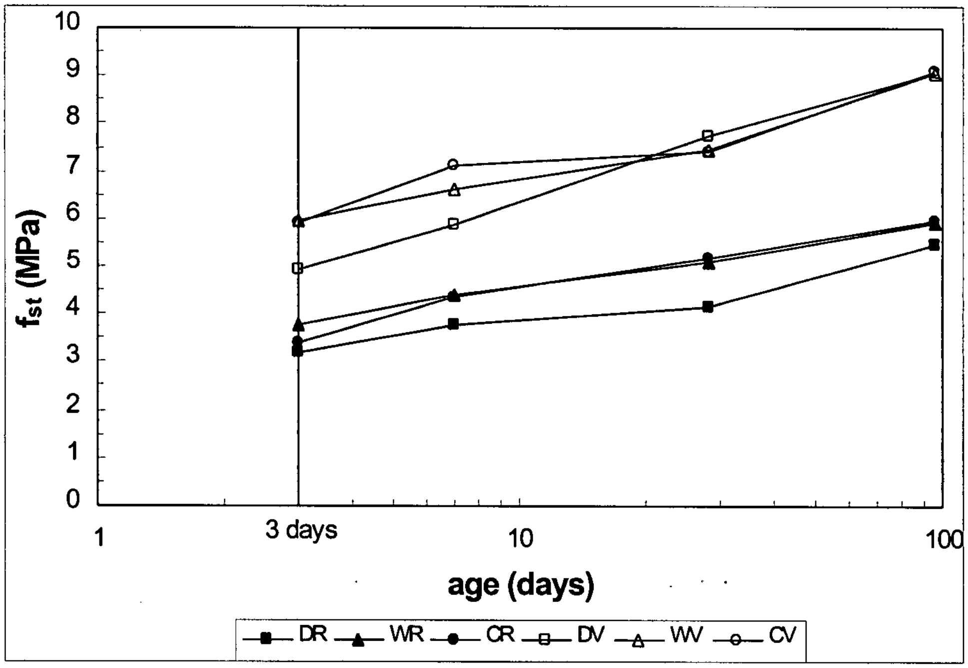


Figure 6.36 Splitting tensile strength vs. age relationship for the different mixes examined (D=dry-mix shotcrete, W=wet-mix shotcrete, C=cast concrete, R=recycled aggregates, V=virgin aggregates).

As far as the production process is concerned, there does not appear to be any particularly major or conspicuous trend depicted by any of the different production processes. The wet-mixes, wet-mix shotcrete and cast concrete, follow almost the same curve for both types of aggregates. This indicates that the neither process has an advantage over the other, in terms of yielding higher tensile strengths, since almost equal strengths are produced at any age. For dry-mix shotcrete, though, lower tensile strengths are obtained with recycled aggregates while, with virgin aggregates, higher tensile strengths are obtained after 28 days. With recycled aggregates, this can be understood since the dry-mix shotcrete had a higher water-cement ratio compared to the wet-mixes; while, with virgin aggregates, its higher gain is not clear.

6.7 Flexural Properties – Pilot Study

6.7.1 Introduction

Shotcrete is frequently subjected to flexural loads during its service life. In applications like tunnel support, rock support or slope stabilization, the applied material will inevitably undergo bending stresses due to the way in which loads are applied or to the possible movement of the supported mass. It is, therefore, important to be able to understand how shotcrete behaves under such conditions, what factors affect its behaviour, and how to incorporate these into design. This is done by performing standard beam tests on specimens cut from a test panel. In recent years, there has been an increased tendency to add short, discontinuous fibers into the shotcrete mix as a replacement or as an addition to reinforcing mesh. The main reason for adding these in sprayed concrete is to impart some toughness to an otherwise relatively brittle material. Fiber reinforcement improves the ductility, energy absorption, impact resistance and crack resistance of shotcrete. Fiber addition enables the shotcrete to continue to carry stresses beyond matrix cracking, which helps maintain the structural integrity and serviceability of structures under load.

The flexural properties of normal fiber reinforced shotcrete have been well documented. In general, the flexural strength can vary from 4-8 MPa, depending on the fiber type and content [57]. The flexural toughness also varies depending on the fiber type and content. The most common methods used to characterize toughness of fiber reinforced shotcrete are ASTM C1018 and the JSCE SF-4. However, there has been much debate over the best method of measurement because the different methods that are currently used are very judgmental and can possibly lead to large discrepancies in results [72].

In this pilot study, the effects of using recycled aggregates, as opposed to virgin ones, in fiber reinforced shotcrete were investigated. In the previous sections, it was discussed how the incorporation of recycled aggregates can lead to reductions in strength, elastic modulus decreases and increases in toughness. This time, it is desired to examine what would occur to the normally measured parameters in fiber reinforced shotcrete. Only wet-mix shotcrete with the same mix proportions as selected mixes was investigated. The same recycled and virgin aggregates were used. The only difference is that an additional 1.0% by volume of 30 mm hooked-end steel fibers was added to the mixes and superplasticizer was added as required to produce a workable and pumpable mixture. Such mechanically deformed fibers have been found to provide the most improvement in bond-slip characteristics as they develop positive anchorage in the concrete [74]. Only the results at 28 days were investigated.

6.7.2 Flexural Strength

The flexural strength, or modulus of rupture, is computed from the maximum load in the flexural load-deformation diagram using an elastic analysis. It is an indication of the maximum amount of tensile stresses resisted at the extreme face of the beam before failure. Thus, for unreinforced specimens, the flexural strength also provides another measure of the tensile strength of the material. This is a matrix property which can be altered by changing, for example, the water-cement ratio or by adding mineral admixtures. In fiber reinforced mixes, two values of flexural strength are significant: the first-crack flexural strength, determined as the strength at the point at which the load-deflection curve first deviates from linearity, and the ultimate strength, corresponding to the peak load. Fibers marginally influence the former parameter, but they do increase the latter parameter as their volume percentage increases.

The results for ultimate flexural strengths of the mixes investigated are shown in Figure 6.37.

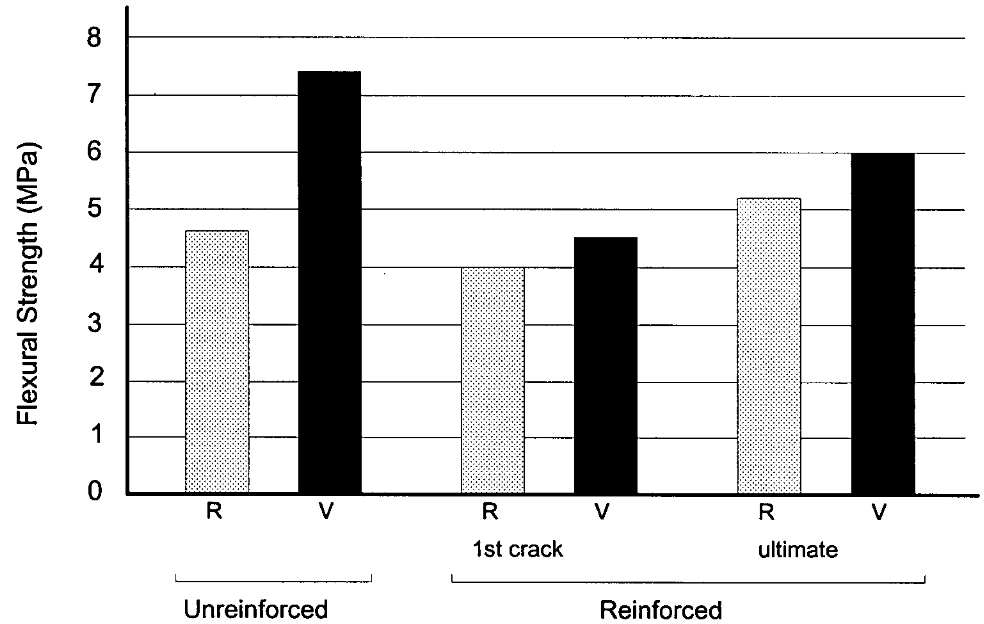


Figure 6.37 Flexural strengths for the mixes investigated (R=recycled aggregates, V=virgin aggregates).

The ultimate flexural strength of 6.0 MPa for the fiber reinforced virgin aggregate mix is similar to results obtained by Banthia using a similar fiber in wet-mix shotcrete [75]. Not surprisingly, the flexural strengths for recycled aggregate mixes are lower than those for virgin aggregate mixes, regardless of whether or not the mixes contained any fiber reinforcement. The modulus of rupture is indicative of the tensile strength of the material, particularly in the plain mixes, where failure occurs at first cracking. It is then easy to understand why recycled aggregate mixes produce lower values for this material property. Based on the discussion of Section 6.6.2, the tensile resistance of cementitious mixes is dependent on the bonds that exist within the various components in the system.

With recycled aggregates, these bonds are not as strong as they would otherwise be with virgin aggregates, due to the presence of dust regions leading to a lack of matrix and interfacial cohesiveness and to the presence of foreign substances which do not possess good bonding capabilities compared to conventional aggregates. Consequently, the tensile strength and, indirectly, the modulus of rupture are hampered as seen from the results.

Considering first the unreinforced mixes, failure takes place at first cracking. This is because, once this first crack occurs at the lower face, there is no mechanism to arrest its propagation; and, as a result, it grows in an uncontrolled manner splitting through the depth of the specimen. The obtained flexural strength, or modulus of rupture, thus is an indirect measure of the tensile strength of the concrete. So, these values can be compared to those obtained from the splitting tensile tests (Section 6.6.2). These values are compared in Table 6.5.

Table 6.5 Indirect tensile strengths for wet-mix shotcrete (28 days).

Aggregate Type	Splitting Tensile Strength (MPa)	Flexural Strength (MPa)
recycled	5.1	4.6
virgin	7.5	7.4

This comparison indicates that the splitting tensile strengths are, at least, as high as the flexural strengths. This does not agree well with what is conventionally known since the flexural strengths are normally higher than the splitting tensile strengths. Flexural strengths are normally higher than splitting tensile strengths because of a smaller volume of material under maximum stress and because of an overestimated stress distribution. The different than expected behaviour could be due to inconsistencies in the batching which produced lower quality panels for this pilot study.

Now, considering the fiber reinforced mixes, as mentioned, there are two types of flexural strengths: the first crack strength and the ultimate strength. The first crack flexural strength occurs when the matrix first cracks and the curve deviates from linearity. If the fibers were not present, the crack would run through the specimen leading to catastrophic failure, as in the case of the unreinforced specimens. So, essentially, these values should correspond to the flexural strengths of the unreinforced mixes; but, as seen from Figure 6.37, this is not the case. The most likely reason for this is that the fibers could have entrained air or voids into the system. The high surface area of the fibers can easily entrap air spaces into the mix. Also, the entanglement of fibers during mixing could lead to the improper spread of paste, leaving behind voids in the inter-fiber regions. At the relatively large volume fractions of fibers used, such air or void entrainment is possible, consequently leading to a more premature matrix cracking. Thus, in fiber reinforced mixes, the matrix is likely to crack at a lower load value than unreinforced mixes. As for the ultimate flexural strength, it occurs when the fibers have reached their maximum load bearing capacity at the bottom face of the beam and start failing either as fiber fracture or as fiber pull-out.

It can be seen from Figure 6.37 that, for both aggregate types, the ultimate flexural strength is higher than the first crack flexural strength which is to be expected for a system that has an adequate level of reinforcement. During the shooting, fibers tend to rotate due to their own inertia and orient in a plane parallel to the surface being sprayed. This behaviour is beneficial in terms of enhancing the flexural properties of the sprayed concrete. One interesting aspect to note regarding the ultimate flexural strength is that, in the virgin aggregate mixes, this parameter is lower than the flexural strength of the corresponding unreinforced mixes; while, on the other hand, in the recycled aggregate mixes, this parameter is higher than the flexural strength of the corresponding unreinforced mixes. In the virgin aggregate mixes, this lower value could be attributed again to the entrapment of voids into the system. This entrapment is so high that not even with the added contribution from the fibers can this strength be as high as the matrix strength itself in the unreinforced mixes. However, in the recycled aggregate mixes, the higher value could be due to the fact that the entrapment of voids in the fiber systems does not significantly add any major internal defects above what the recycled material has already caused in the system. In other words, the recycled aggregate systems have so many defects to start with, that any additional defects would be relatively insignificant and thus, the strengthening imparted by the fibers is more pronounced. On the other hand, with virgin aggregates, the entrapment of voids into a relatively homogeneous and defect-free system can be detrimental enough to counteract the benefits provided by the fibers. And, finally, the reason why fiber reinforced recycled aggregate mixes have a lower ultimate flexural strength than fiber reinforced virgin aggregate mixes is because the fibers cannot develop their full bonding potential in the former type of mix. This is probably caused by an alteration in the matrix which hinders it from possessing good bonding characteristics with the fibers, and the dust is likely to be this factor. This would lead to the speculation that fibers pull out from the matrix, which is indeed what was observed in the fractured specimens. It will be interesting to confirm some of these trends through single fiber pull-out tests.

From the above observations and discussion, it can be concluded that, although recycled aggregate mixes have lower flexural strengths compared to virgin aggregate mixes, the effects of fiber additions are more pronounced in recycled aggregate concrete. So, fibers can be more effectively used in mixes with recycled aggregates in terms of enhancing flexural strength. As will be discussed in the next section, it is the flexural toughness that gets enhanced the most by adding such discontinuous fibers to the mix.

6.7.3 Flexural Toughness

Flexural toughness is generally recognized as the property most enhanced by the addition of fibers in concrete and shotcrete. The enhanced performance of fiber reinforced mixes over their unreinforced counterparts comes from improved capacity to absorb energy during fracture. While a plain unreinforced matrix fails in a brittle manner at the occurrence of cracking stresses, the ductile fibers in fiber reinforced concrete continue to carry stresses well beyond matrix cracking, helping maintain structural integrity and cohesiveness. This added performance is reflected in the load-deflection diagram by a gradual softening of the material after the peak load has been reached, as shown in Figure 6.38 for the virgin aggregate mixes. The two most common methods currently employed to characterize the toughness are ASTM C1018 and JSCE SF-4. Details of the analyses for each method are outlined in References 76 and 77, respectively. Although there has been considerable debate over the reproducibility and accuracy of these methods, if the specimen and testing conditions are maintained the same, for the purposes of this experiment, these characterization methods will provide an indication of the relative performance between mixes with recycled aggregates and mixes with virgin aggregates.

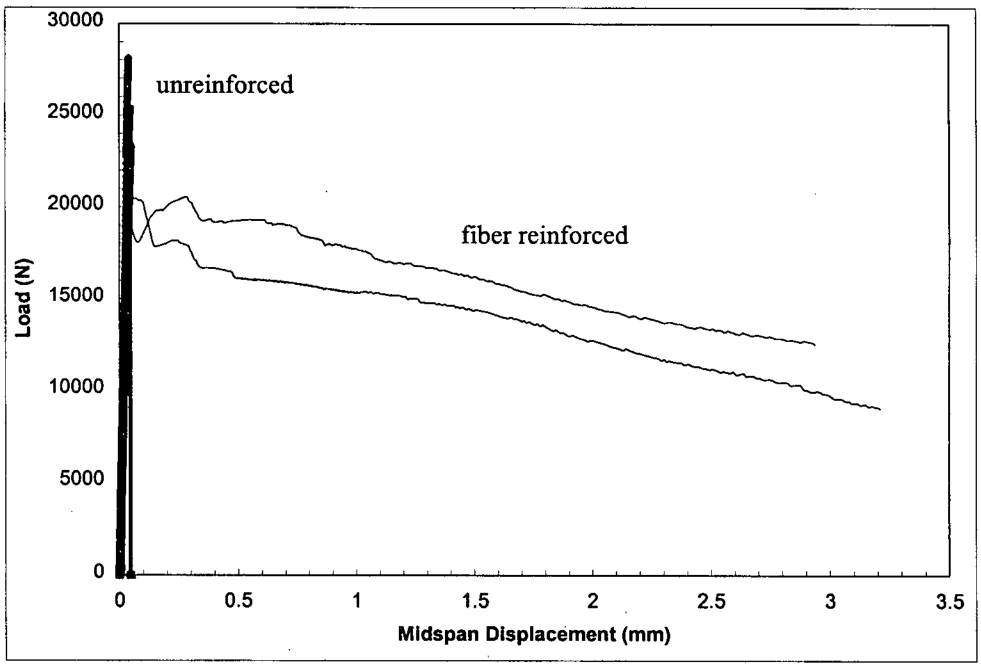


Figure 6.38 Load-deflection diagrams for virgin aggregate beams (plain and fiber reinforced).

To compare the performance between fiber reinforced mixes with recycled aggregates and mixes with virgin aggregates, it is helpful to look at the complete load-deflection diagrams for these two types of mixes. This is shown in Figure 6.39. First of all, a considerable amount of post-peak load carrying capacity exists in both types of mixes as

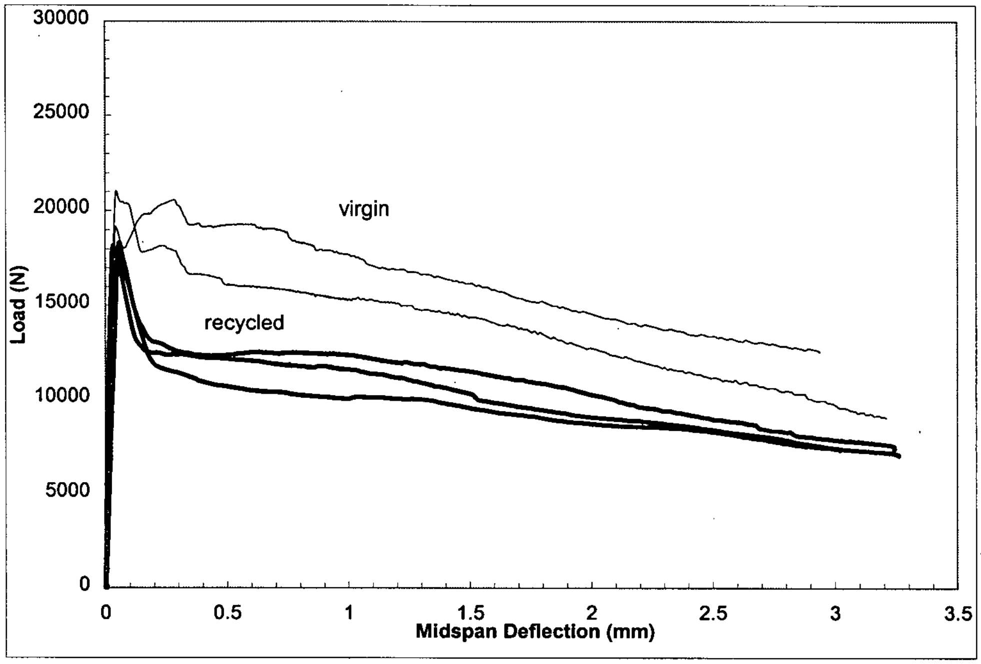


Figure 6.39 Load-deflection diagrams for fiber-reinforced mixes (recycled and virgin aggregates).

a result of the addition of fibers. The mixes containing virgin aggregates have a significantly higher load carrying capacity, including right after the peak load has been reached, and gradually this load carrying capacity reduces with increasing midspan deflection. In one of these beams, even some strain-hardening is observed. On the other hand, the mixes containing recycled aggregates all undergo strain-softening after the peak point is past. The causes for some of these trends will be discussed later; however, it is important to present the quantitative analysis of these load-deflection plots in order to derive quantities suitable for a general comparative assessment of the different aggregate types. It should be kept in mind when looking at the values that, for both aggregate types, the in-situ fiber content was around 0.90% with the recycled aggregate mix having a slightly higher value. The results of the quantitative analysis as determined from the two techniques, ASTM C1018 and the JSCE SF-4, are summarized in Table 6.6.

Table 6.6 Summary of results for the analysis from the load-deflection diagrams.

		First crack parameters			ASTM Indices								Japanese Indices	
Aggregate Type	Sample	Deflection (mm)	Strength (MPa)	Toughness (Nmm)	I_5	I_{10}	I_{20}	I_{30}	I_{50}	$R_{5,10}$	$R_{10,20}$	$R_{20,30}$	$T_{L/150}$ (Nmm)	F_b (MPa)
Recycled	1	0.021	4.4	175	5.0	9.4	17.3	24.7	39.2	88	79	74	22729	3.3
	2	0.037	3.8	234	6.6	11.8	21.8	31.6	51.4	104	100	98	24069	3.5
	3	0.032	3.8	224	6.5	10.8	19.1	26.8	51.8	86	83	77	20754	3.0
	Avg.	**0.030**	**4.0**	**211**	**6.0**	**10.7**	**19.4**	**27.7**	**44.1**	**93**	**87**	**83**	**22517**	**3.3**
Virgin	1	0.031	4.6	245	6.0	11.7	22.6	32.8	52.5	114	109	102	30816	4.4
	2	0.030	4.3	224	5.7	11.9	24.9	37.3	62.1	124	130	124	34748	5.0
	Avg.	**0.030**	**4.5**	**235**	**5.8**	**11.8**	**23.8**	**35.1**	**57.3**	**119**	**120**	**113**	**32782**	**4.7**

From the results in Table 6.6, it can be seen that, although recycled and virgin aggregate mixes have similar first crack parameters, the post-peak properties are fairly different. In general, the virgin aggregate mix has higher energy absorption capacities at all deflection levels, as represented by larger toughness indices at the specified locations and a higher cumulative toughness at L/150. Also, virgin aggregate mixes have a higher residual strength factor compared to the recycled aggregate counterparts. In terms of percentage reduction, the mix with recycled aggregates has reductions of the indices in the order of 10-30%. These reductions can be readily seen from the load-deflection curves of Figure 6.39 by noting that the curves for the recycled aggregate mix lie well below those for the virgin aggregate mix. To understand why such reductions occur for the mix with recycled aggregates, it helps to understand what mechanisms are present after the peak load that provide the desired post-peak load carrying capacity. After the matrix has cracked, the dominant load sustaining mechanism in the system is provided by the failure of fibers. Fiber fracture leads to brittle behaviour and a quick drop in the load carrying capacity, but this was not observed here. Fiber pullout occurs after the matrix-fiber interfacial bond strength has been exceeded and the fibers start to be withdrawn from the matrix. If there exists any mechanical deformation of the fibers, some kind of anchorage with the matrix can be expected in the form of bearing resistance. The continuous pull out of fibers along with breakage of the matrix due to fiber anchoring forces for all the fibers bridging the crack opening as the crack propagates are what provides the beam with its post-peak load carrying capacity. So, in a case where fibers can pull out easily or where the matrix cannot resist the crushing forces, the sustainable load would be less.

These kinds of inferior mechanisms do exist in recycled aggregate mixes which makes it reasonable to expect that this kind of mix would not be able to support as much load because of the reduced pull-out resistance that fibers have in the weakened matrix. In terms of developing a strong matrix-fiber interface, there is a limit to which this bond strength can develop. Just as a matrix containing recycled material cannot bond well to aggregates, this same matrix is not likely to bond well to fibers either. The large amounts of dust coming from the fine material in the system interfere with the formation of sound bond. Besides the oriented growth of calcium hydroxide in the interfacial region and the larger quantities of ettringite there, the dust can form aggregations of inert particles on the fiber surface, which impede the penetration of cement paste into voids and onto the fiber surface. Thus, unbonded regions exist on the fiber surface which reduce the average interfacial strength, as well as providing less resistance to fiber pull-out. In terms of the matrix crushing strength, fine recycled material also hampers this property. It was seen in earlier sections how the compressive strength was affected by a poorer matrix quality. This weaker matrix would also provide less resistance to a fiber in the process of pulling out, even if the fiber is deformed. Therefore, putting all these factors together, it can be easily seen why fibers can pull out more easily in a mix with recycled aggregates, as opposed to in a mix with virgin aggregates. Consequently, the post-peak energy absorbing capacity of fiber reinforced recycled aggregate mixes is inferior to mixes made with virgin aggregates.

The lack of proper bonding between a matrix with recycled fines and fibers can also explain another trend seen in the load-deflection curves of Figure 6.39. While virgin aggregate mixes reached peak load and passed it in a stable manner, recycled aggregate mixes displayed some instability as soon as the peak load was passed. It can be seen from the curves that the peak loads for these mixes are around 18,000 N but quickly drop to about 13,000 N before stability is regained. This sudden drop in the load is what makes recycled aggregate mixes yield lower toughness indices. Obviously, this behaviour is not due to matrix strength since the recycled aggregate matrix is weaker than the virgin aggregate matrix. So, it must be due to how quickly the fibers can take over in carrying the applied load. At peak load, the load is carried by an assortment of mechanisms within the system; but, once this has been passed, the load is almost completely transferred to the fibers. It appears that the partial instability in the recycled aggregate mixes is due to the slippage of the fibers when the load is suddenly transferred to them at the critical crack opening displacement. Since the matrix-fiber bond is not strong enough, the interfacial bond is broken very easily and the fibers can slip a certain amount before their anchorage takes effect. After this partial instability, the recovery in load carrying capacity is then probably due to the mechanical anchorage of the fibers that slipped at the critical moment and to the onset of new pull-out processes of previously intact fibers. This kind of behaviour does show that the attainable bond between a recycled aggregate matrix and fibers is very fragile. The post-peak load carrying capacity may then be due entirely to the anchorage of the fibers. As such, straight fibers with no mechanical or surface deformation are likely to fail very quickly and in an unstable manner if used in recycled aggregate mixes. Perhaps, a highly deformed fiber can be designed only for recycled aggregate shotcrete.

In civil engineering works, the shotcrete applied must meet certain performance specifications in order for it to be acceptable. Morgan provides specifications, which include minimum requirements for the toughness indices, as shown in Table 6.7 [79]. Besides the setting time tests and the early age strength tests which were not carried out in the experiment, the fiber reinforced recycled aggregate mixes come very close in meeting all these requirements. The properties related to strength are all well satisfied by such mixes. Recall from Section 6.2.5, the 28 day compressive strength of these mixes are about 42 MPa which is over what is required. This is as a result of the fibers actually providing compressive strength enhancement to the mix, at the proportions used. The 28 day flexural strength for the recycled aggregate mixes is 5.2 MPa which is slightly below the limit but still tolerable. On the other hand, the toughness indices are well above the values required by such typical specifications. Thus, although fiber reinforced recycled aggregate mixes may not be as strong or as tough as their virgin aggregate counterparts, they do provide performance levels which are acceptable and beyond those required in typical shotcrete applications. The only other concern would be the much higher boiled absorption and volume of permeable voids of these mixes, which may hamper their long term performance. Therefore, as mentioned in the previous sections, it is important to perform durability tests on these mixes and to investigate the underlying mechanisms taking place because, for these recycled aggregate mixes, the 28 day performance may not be totally indicative of its long term performance.

Table 6.7 Typical performance specification for steel fiber reinforced shotcrete [79].

Property	Test Method	Minimum Requirements
Accelerated setting time	ASTM C403	
initial set		10 minutes
final set		30 minutes
Compressive strength, MPa	ASTM C42	
8 hour (accelerated)		5
1 day		10
7 days		30
28 days		40
Flexural strength, MPa	ASTM C78	
7 days		4
28 days		6
Toughness index	ASTM C1018	
I_5		3.5
I_{10}		5.0
I_{30}		14.0
Boiled absorption	ASTM C642	8% maximum
Volume of permeable voids	ASTM C642	17% maximum

6.8 Microscopic Observations

From the foregoing sections, it was seen how recycled aggregate mixes performed differently to virgin aggregate mixes under different loading conditions. The general trend seen was that of a weaker composite with more deformation capacity. This was manifested from the increasing nonlinearities in the stress-strain curves, the lower compressive strengths, the lower splitting tensile strengths, the lower elastic moduli, the lower flexural strengths, the higher strains at peak load, and the higher energy absorption capacity of such mixes. At the same time, while these mixes did not display a pronounced gain in strength past 28 days of age, they did portray an increasingly larger ability to deform past this age. Moreover, the production process employed did not seem to have much effect on performance. Hence, it was postulated that some kind of mechanism must exist in recycled aggregates systems which deters them from fully acquiring their strength potential while rendering the systems much softer and ductile. These mechanisms allow cracks to grow in a less resisted manner thereby, enabling them to quickly spread throughout the entire composite. An examination of the raw recycled aggregates indicated that there were large amounts of dust in the dry-mixed crushed waste concrete. It was assumed that this dust would be incorporated in the hardened mix forming regions where cement paste cannot penetrate thus, leaving voids behind. Of all the performance weakening factors in recycled aggregate mixes, such as dust presence, lack of proper bond with old aggregates and old matrix, increased porosity, contaminants and soft materials, it appears that the existence of excessive dust is the most serious detriment. This can be observed by noting the vast quantities of dust in the recycled aggregates and quantitatively measured by noting the large percentage reductions in various physical properties compared to those systems reported in the literature which had no recycled fine material. To visually witness this factor, observations of the material under a scanning electron microscope were done of the matrix alone and at the interface with aggregates.

Figure 6.40 shows a typical picture of the matrix made with virgin aggregates while Figures 6.41 shows a typical picture of a matrix made with recycled aggregates. Both pictures were taken at the same magnification. It can be seen that the virgin aggregate matrix is fairly dense, compact and homogeneous. There do not appear to be many large pores even at this large magnification. Some calcium hydroxide crystals can be seen but these tend to be scattered and of limited size. The dense background is calcium silicate hydrate which appears as a continuous uninterrupted phase. On the other hand, the recycled aggregate matrix appears as a very loose, unconsolidated and nonuniform material. It can be seen that plenty of inter-particle voids exist in this material, some of which even look as if they were large cavities. This matrix contains a significant proportion of loose debris which is likely to be dust. This seems reasonable considering that the dust particles fall in the size range of a few tens of microns. Some of the loose debris may also be large crystals of calcium hydroxide which found enough room to grow.

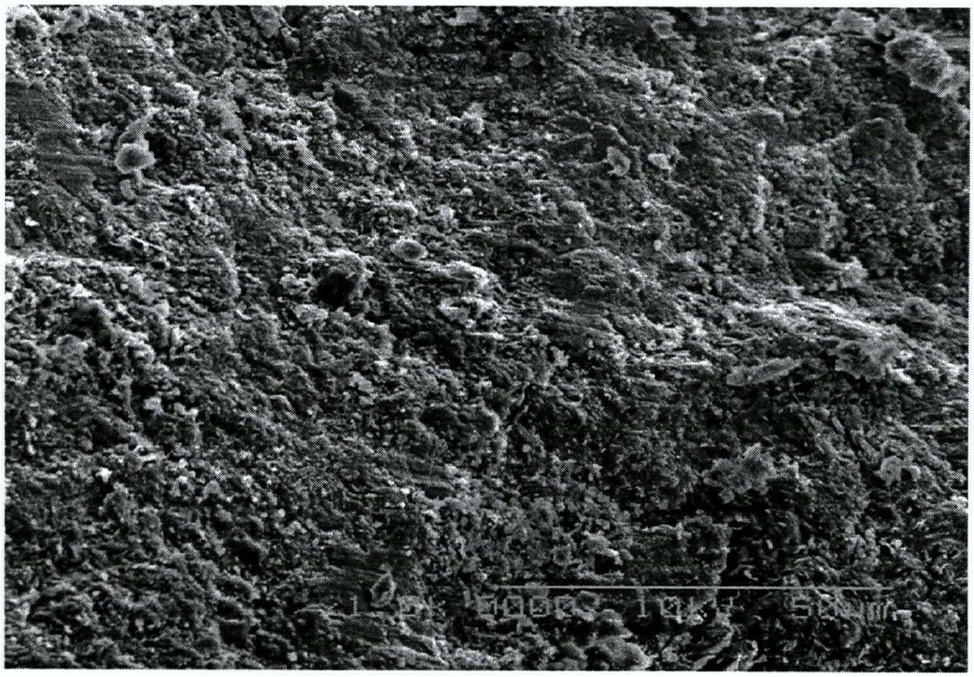


Figure 6.40 Virgin aggregate matrix.

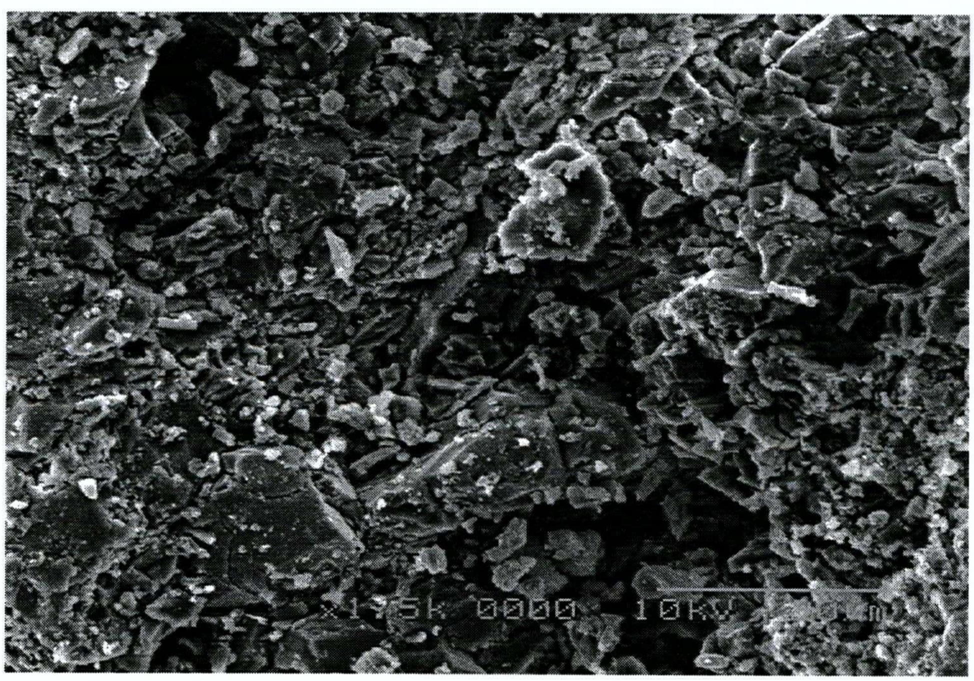


Figure 6.41 Recycled aggregate matrix.

When both types of matrices described above are loaded, it is obvious to expect that the matrix made with virgin aggregates would provide much more resistance to a propagating cracks compared to the matrix made with recycled aggregates. The homogeinity and low porosity of the virgin aggregate matrix would likely possess higher adhesive and cohesive forces, which forms a tough medium through which cracks must force their way. On the other hand, the loose nature of the particles in the recycled aggregate matrix can be pictured as one that crumbles more readily under an applied load. These particles are not very well attached to each other and sliding can easily occur within the material giving it an overall larger deformability. As a result of this type of internal structure, moving cracks find very little resistance to their growth in such matrices. The material would thus fail at lower loads and at higher deformations. The loose particle nature of this material was observed from the fractured specimens as these samples broke in numerous pieces and the broken pieces contained lots of dust, whereas virgin aggregate specimens just broke in a few pieces leaving behind no loose material.

Considering now the matrix-aggregate interface, Figure 6.42 shows a typical interfacial region from a concrete made with virgin aggregates while Figure 6.43 shows a typical one from a concrete made with recycled aggregates. In Figure 6.42, the solid, dense and partly striated phase on the upper half of the picture is the aggregate. The more porous material attached to it is the interfacial region. This region appears more porous than the matrix away from this zone, as shown in Figure 6.40, because interfaces tend to be more porous and contain more material of oriented growth. On the other hand, the interface in a recycled aggregate concrete as depicted in Figure 6.43 shows a significant proportion of large and loose particles. These particles do not appear to be cohered in any manner but just appear as if they had been accumulated there. There seems to be some kind of "wall-effect" in the packing of these dust particles against the aggregate surface. Also, from the picture, it can be seen that there is no cement paste penetrating into the void regions of the dust particle conglomeration. From this, it is evident that the interfaces in concretes made with recycled aggregates are not very well bonded and contain regions of inefficient particle packing, leaving behind clumps of loose particles.

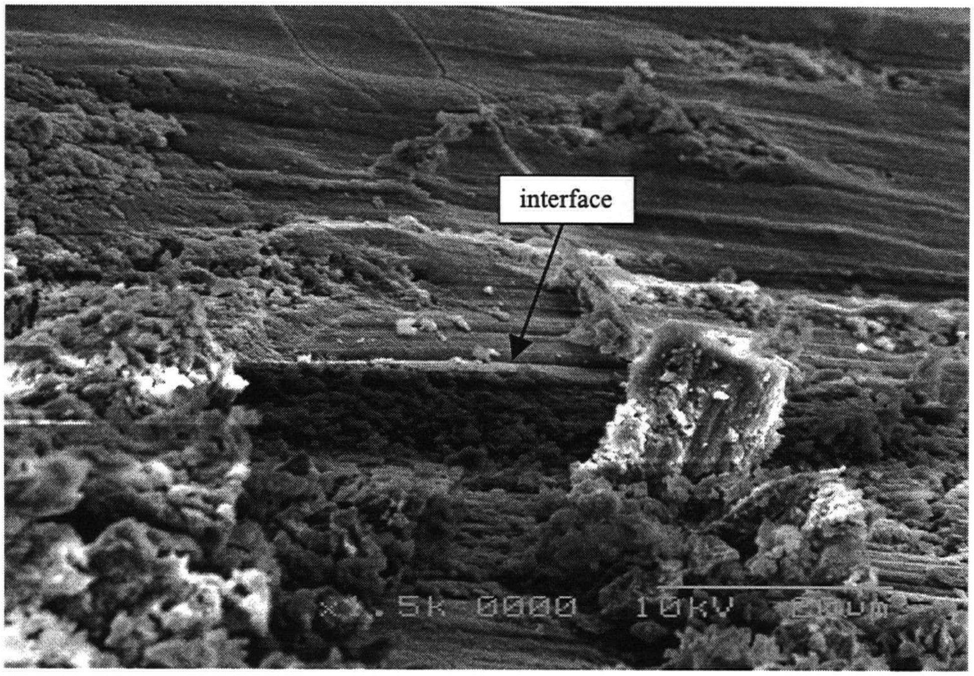


Figure 6.42 Virgin aggregate interface.

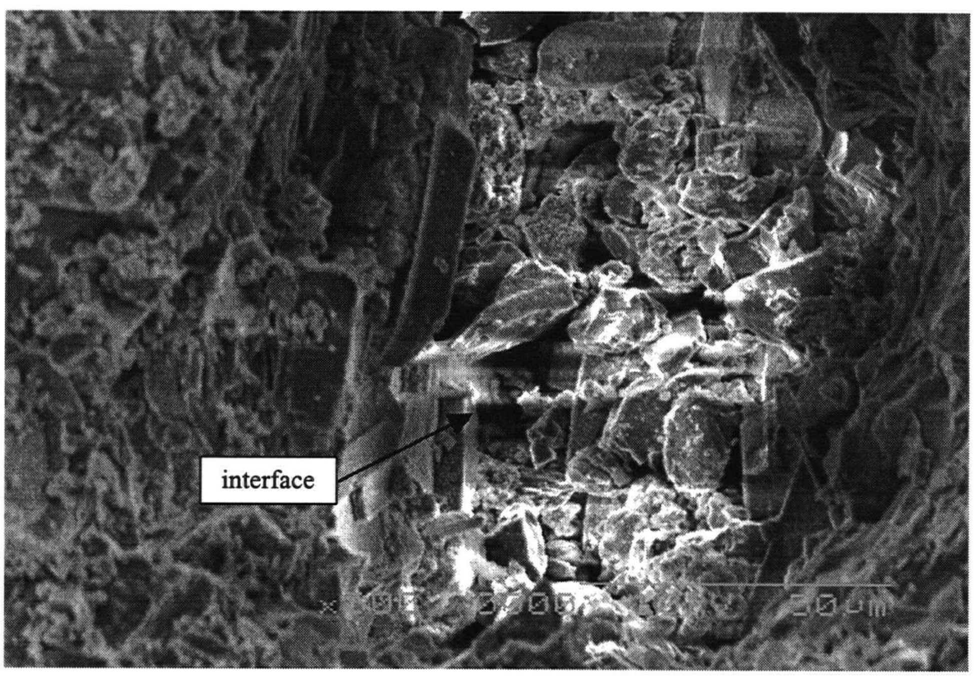


Figure 6.43 Recycled aggregate interface.

When both types of interfaces described above are loaded, it is reasonable to expect that the one made with recycled aggregates would undergo debonding in a fairly easy manner. With very little paste holding the loose particles together and connecting them to the aggregate surface, a sound interfacial region is almost nonexistent. Since most of the failures in concrete start at the interface, recycled aggregate systems would suffer from premature failures starting as a quick matrix-aggregate separation at the aggregate surface followed by the formation of cracks which propagate with little resistance through a weak matrix like that shown in Figure 6.41. Much of this failure process occurs with the movement and sliding of the loose particles.

From these SEM pictures, it can be seen that, indeed, in a system made with recycled aggregates, abundant levels of dust exist either at the matrix or at the interface. Such impurities render the material less cohesive and easier to fracture. Since cracks can propagate at lower load levels and there is a significant amount of frictional sliding in regions that are unbonded, the overall material experiences higher stress-strain nonlinearities which lead to lower peak loads, but higher ductilities.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Final Remarks and Conclusions

An experimental study was conducted to investigate the feasibility and applicability of using crushed waste concrete from a local crushing plant as recycled aggregates for the production of concrete and shotcrete. The objective was to determine the suitability of recycled aggregate as a material and its effect in concrete from both fresh and hardened properties.

Suitability as Aggregates

The properties and composition of the crushed waste concrete can vary from batch to batch, depending on the source of the demolished concrete, the types and amount of processing involved, and the nature and levels of contaminants present. Properties of these must therefore be evaluated for every new batch. In general, recycled aggregates were determined to have a slightly lower unit weight (1720 kg/m^3) and relative density (2.48) and a much higher water absorption (3.7% for the coarse portion and 12% for the fine portion). The particle size distribution can normally be adjusted to suit specific applications and a method for blending aggregates of different gradations was detailed in this thesis. Another serious consideration when dealing with recycled aggregates is the presence of foreign substances passed along from demolition as these provide the aggregates with a number of undesirable physical and chemical characteristics.. Short term effects such as high water absorption, gradation and density can be accounted for by some additional processing and by adjusting the mix proportions. Long term effects as caused by the various chemical instability of the contaminants were not covered in this experiment but must be studied for a more complete evaluation of the material.

Effects on the Fresh Mix

In a fresh mix, the higher absorption of recycled aggregates leads to a premature stiffening and a rapid loss of workability of the mix, therefore requiring continuous mixing and vibration. Adjustments must therefore be made to compensate for the additional water demand. Besides this, the production of cast concrete with recycled aggregates is very similar to the production of concrete with virgin aggregates. When used in shotcrete, the angular and coarse nature of recycled aggregates may cause partial hose blockages, experienced as pulses of material as it is being delivered. On the other hand, the higher cohesiveness of this material leads to lower rebound values in both, material (3-4% less) and fiber (7% less). The in-situ material would thus have a more uniform aggregate gradation as well as a higher content of fibers.

Effects on the Hardened Mix Properties

The stress-strain curves for recycled aggregate mixes depict that such materials possess very stable loading and unloading behaviour. The compressive strengths of these mixes are in the range of 40-56% lower than the corresponding strengths achieved with virgin aggregates because of a weaker internal structure. This is true regardless of the production process used. The strength gain behaviour does not differ much from that of mixes with virgin aggregates. However, recycled aggregate mixes support higher strains at peak load, and this increases with increasing age of the specimen. Thus, the material can sustain larger deformations before losing load carrying capacity. From an energy absorption standpoint, recycled aggregate mixes have much higher energy absorbing capabilities than their virgin aggregate counterparts. This can be seen from, either, absolute energy values or post-peak energy ratios. Such behaviour manifests the increased ductility of recycled aggregate mixes. These mixes also display lower modulus of elasticity values, in the range of 36-48% lower than their virgin aggregate counterparts. A new coefficient in the modulus of elasticity versus compressive strength relationship ($E = K (f_c')^{1/2}$) was thus determined for recycled aggregate mixes. This is found to be 3822 (in MPa) and 46,000 (in psi) at any age. A damage quantification parameter can also be used to track the extent of damage incurred in the system. This reveals that recycled aggregate mixes suffer from more damage than virgin aggregate mixes and this effect continues at later ages. The splitting tensile strength for recycled aggregate mixes is in the range of 30-47% lower than those for virgin aggregate mixes. However, because of the lower sensitivity of the tensile strength to recycled aggregate additions, compared to compressive strength, it is suggested that there must be some mechanism in recycled aggregate mixes that seriously impairs the matrix performance causing a larger reduction in compressive strength. Steel macro-fiber reinforced shotcrete containing recycled aggregates have somewhat inferior flexural properties compared to shotcrete with virgin aggregates. However, at the fiber volume investigated, the flexural strength and post-peak toughness of recycled aggregate mixes is at levels adequate enough for normal shotcrete applications.

In general, when dealing with recycled aggregates, some of the rules traditionally used in ordinary concrete cannot be used for concrete with waste material. New relations must be determined. The 28 day performance cannot be used as a reliable indication of material quality. Moreover, compressive strength cannot be used as a sole indicator of material quality, including durability. Hence, care must be exercised when applying conventional beliefs o recycled aggregate systems.

At present, the use of recycled aggregates in concrete appears to be limited only to applications where the demand on material performance is not so strict. However, with an increasing amount of waste concrete being produced every year and in every nation, this problem must be addressed with efficacy and without hesitation by every participating member in the industry.

7.2 Recommendations for Future Research

Throughout the course of the research and based on the observations gathered, a number of areas were prompted as ones for which extra research is possible and needed. Following is a list of suggestions for future work in this subject.

1) It would be interesting to investigate whether or not similar percent reductions in performance would occur if the aggregates were not dry-mixed as they were in the experiments. Also, it would be interesting to see the effects of removing the dust from the aggregates by, perhaps, water washing.
2) For a complete evaluation of the crushed waste concrete as aggregates, it is recommended that physical and chemical durability tests be carried out on these. Physical durability tests would include abrasion and frost resistance tests, while chemical durability tests include the potential for alkali-aggregate reactions and the presence of reactive substances like sulphates.
3) Since shotcrete containing recycled aggregates is more cohesive, leading to lower material rebound values, much like the way silica fume does, it is worth investigating the amount of build-up this material can have during shotcreting. It is expected that this material can achieve higher build-ups compared to a normal aggregate shotcrete.
4) Another possible investigation to carry out on the rheology of such mixes is to make use of air-entraining agents in the wet-mix shotcrete. The dispersed air bubbles throughout the mix will provide more lubrication and possibly help eliminate the hose blockages that were experienced during the shootings. At the same time, greater build-ups with even lower rebounds may be achieved.
5) It is strongly recommended to carry out investigations on the use of mineral admixtures in recycled aggregate mixes. Given the large amounts of dust in the dry-mixed aggregates, most of which is comprised of pulverized old mortar, it is well speculated that large quantities of calcium hydroxide and unhydrated cement exist there in fine form. A mineral admixture containing high levels of silica would potentially induce significant additional hydration reactions that will

enhance the cementitious nature of the system. In this respect, silica fume is perhaps the ideal candidate; however, extra caution must be placed on the workability of such mixes.

6) Since the energy absorbing capacity of recycled aggregate mixes is far greater than that of virgin aggregate ones, it is advisable that full scale impact tests be carried out on this type of mixes.
7) If recycled aggregates are to be employed extensively in the concrete industry, it is likely that its domain of application would eventually span into structural applications. For this reason, it is well advised that reinforced concrete made with recycled aggregates be investigated. In particular, it would be interesting to see how the current reinforced concrete design codes would have to be modified to account for the possible changes that could occur when such material is used.
8) To determine the bonding qualities of the recycled aggregate concrete to fibers, it is recommended that pull-out tests be carried out with a twin-cone type of fiber and the results be compared to those obtained with a hooked-end type of fiber.
9) An interesting investigation would be the use of micro-fibers in recycled aggregate systems. Micro-fibers have the ability to arrest microcracks thereby delaying their growth and possibly leading to higher material strengths. However, once the cracks grow to significant sizes, they are bound to undergo the same mechanisms as an unreinforced mix and thus, the energy absorbing mechanisms are still existent. It may be possible to observe a system with higher strengths and, at the same time, ductile.
10) Perhaps the most important recommendation out of all is to conduct extensive experimentation on the durability and durability related aspects of recycled aggregate systems. With the growing trend towards sustainable concrete systems for longer lasting structures, durability becomes an important issue aside from structural design. In the case of recycled aggregate systems, this has more of a relevance as it is well-known by now that such systems have limited room for

strengthening, yet have the potential to self-deteriorate from the internal chemical reactions that could occur and due to their higher porosity.

Specimens made with recycled aggregates should be kept for long periods of time and then tested to see what kind of changes occur through that time. Probably the simplest test is for the compressive strength which would give an indication of the overall condition of the material. However, other properties that should be considered and tracked with time are its freeze-thaw resistance, its permeability, its abrasion resistance, and its sulphate and carbonation vulnerability.

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