Salt scaling resistance – the effect of curing and one-week presaturation

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Abstract

Salt scaling resistance – the effect of curing and one-week presaturation

Reza Mohammadi Ahani

In cold climates de-icing salts (NaCl, $CaCl_2$) are regularly used to prevent the freezing of water on the horizontal surface of concrete structures like roadways and walkways which results in salt scaling, one of the major durability issues of the concrete.

In this experimental research, the effect of one-week pre-saturation and three curing methods (standard 14-day moist curing, three-day curing, and the usage of a curing compound) on salt scaling resistance was studied in three concrete types including the plain concrete and concretes made with fly ash and slag at 25% and 35% cement replacement, respectively. Two salt scaling test methods used in this research included ASTM C 672/C 672M and its proposed replacement method (one-week presaturation).

One-week pre-saturation was observed to improve the salt scaling resistance of the plain concrete and slag concrete while decreasing the resistance of fly ash concrete.

The application of the curing compound resulted in lower salt scaling resistance than the standard 14-day moist curing method in fly ash and slag concretes and higher resistance in the plain concrete. The three-day moist curing method showed higher resistance than the standard 14-day moist curing method in fly ash concrete, lower resistance in slag concrete, and similar resistance in the plain concrete.

The concrete made with slag showed the highest salt scaling resistance in accordance with ASTM C 672 and its proposed replacement method followed by the plain concrete followed by the concrete made with fly ash.

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Chapter 1

Introduction

1.1 Background

Concrete is the most widely used building material in the world which is a composite material consisting of coarse and fine aggregates, Portland cement, and water. The role of aggregates in the concrete is providing the stiffness and reducing drying shrinkage (Valenza II, 2005) while the paste, the mixture of cementitious materials and water, can be thought as an adhesive that binds the other materials together where the cement has reacted and set. Fresh and hardened properties, in addition to performance in service and costs, are factors in determining a suitable mixture of the component ingredients. Where needed, chemical admixtures including water-reducers and air-entraining admixtures may be also added to the concrete mixture. In some applications, part of cement may be replaced by supplementary cementing materials like silica fume, fly ash, or ground granulated blast furnace slag.

One of the most important issues regarding concrete is the durability of concrete structures which results in high maintenance costs. For example, salts (NaCl, $CaCl_2$) are regularly used in cold climates to avoid the freezing of water on the horizontal surface of concrete structures like roadways and walkways. The application of salt on the concrete surfaces results in salt scaling, one of the major durability issues of the concrete which should be prevented. Salt scaling is defined by Valenza II and Scherer (2007a) as the "superficial damage caused by freezing a saline solution on the surface of a concrete

body" which can advance into concrete over time (Pigeon and Pleau, 1995) and takes place by the removal of "small flakes or chips" of the paste (Arnfelt, 1943; Jacobsen, 1995) with the removal of very few small aggregates (Jacobsen, 1995). The first observation of salt scaling was reported through laboratory tests by Arnfelt (1943) as well as Verbeck and Klieger (1957) which was later verified by Jana (2004) through field tests.

Generally, salt scaling is not visually pleasing because of the exposure of the coarse aggregate. Also, the surface of a scaled specimen would be more sensitive to the penetration of fluids and detrimental ions like chloride which results in the corrosion of the reinforcing steel and concerns the durability of the concrete body (Valenza II and Scherer, 2007a).

A list of characteristics of scaling damage is presented in Chapter 2. Finding a way to prevent salt scaling is not so easy because there are different causes for this damage (Valenza II and Scherer, 2007a). Several mechanisms for salt scaling have been proposed but only the glue spall mechanism was found to account sufficiently for all the characteristics of salt scaling damage which are given in Chapter 2 (Valenza II and Scherer, 2007b). The other mechanisms for salt scaling proposed by several researchers include internal crystallization (hydraulic pressure, crystallization pressure), role of salt (thermal shock, precipitation and growth of salt, salt concentration in the pore solution (reduction in vapour pressure, osmotic pressure)), diffusion theory, supercooling, Litvan's model, and Solar effects. These mechanisms are further defined in the next chapter.

There are several test methods regarding salt scaling including ASTM C 672/C 672M (2003), the capillary suction of de-icing chemicals and freeze-thaw test (CDF) (Valenza II and Scherer, 2007a), the Swedish Standard SS 13 72 44 (SIS, 1992) also known as Borås method, BNQ NQ 2621-900 (2002), and the proposed replacement method for ASTM C 672.

Parameters which influence the salt scaling resistance of a concrete surface include the role of supplementary cementing materials (silica fume, fly ash, ground granulated blast furnace slag, ternary blended cement) and their replacement amount, the effects of different curing methods, entraining air and spacing factor, field exposure, saline solution, finishing, compressive strength, water to cement ratio, bleeding, late exposure to the freezing-and-thawing cycles, maturity, minimum temperature, freezing rate, osmotic pressure, and polypropylene fibre. These parameters and their effects on salt scaling resistance are described in detail in Chapter 2.

1.2 Objective and scope

The main objectives of the present research are:

- Comparing the current ASTM C 672/C 672M (2003) with its proposed replacement method for three concrete types (the plain concrete and the concretes made with fly ash and slag at 25% and 35% replacement of cement, respectively)
- Studying the effects of three different curing methods (the standard 14-day moist curing method, the three-day curing method, and the usage of a curing compound) on salt scaling resistance

The experimental program consists of testing two specimens for each variable studied in this research regarding the salt scaling. Totally three different concrete types were produced including the plain concrete and the concretes made with fly ash and slag at 25% and 35% replacement of cement, respectively, but due to the mixer capacity two mixtures for each type were cast. After mixing each concrete mixture, the slump, the unit weight, and the air content were determined according to ASTM standards followed by the compression tests on each of three cylinders from each concrete mixture at the age of 28 days. One batch of each concrete mixture was used to compare the ASTM C 672/C 672M (2003) with its proposed replacement method while the second batch was used to study the effects of the three-day curing method and the usage of a curing compound on salt scaling resistance of concrete surfaces.

1.3 Outline of thesis

In the following chapter, a detailed literature review regarding the salt scaling is discussed. In this chapter, the proposed mechanisms for salt scaling, different salt scaling test methods, and all parameters influence the salt scaling resistance of a concrete surface are explained.

A detailed description of experimental program and two different salt scaling test methods used in this study is presented in Chapter 3. This chapter also includes the specification of the materials, various tests conducted on the aggregates (sieve analysis, specific gravity, absorption, bulk density, and the moisture content), the properties and mix proportion of concrete, fresh concrete tests (slump, unit weight, and the air content), compression test, and the description of three different curing methods (the standard 14-

day moist curing method, the three-day curing method, and the usage of a curing compound).

The experimental results are presented and discussed in Chapter 4. First, the results of the tests conducted on the fresh concrete including the slump, the unit weight, and the air content are presented. Then the compressive strengths of different concrete mixtures made in this research are given. Finally, the results of salt scaling test are presented which include the visual rating and the relationship between mass loss and the number of freezing-and-thawing cycles. Based on the experimental results of the salt scaling (mass loss), a comparison of ASTM C 672 and its proposed replacement method for each concrete type, the effect of different curing methods on salt scaling, and a comparison of three concrete types according to each variable studied in this research is demonstrated and discussed in this chapter.

The final chapter includes a summary of the conclusions and recommendations for the further research in the area of salt scaling as well as the discussion of the results.

Chapter 2

Literature review

2.1 General

The application of salts (NaCl, $CaCl_2$) on the horizontal surface of concrete structures like roadways and walkways is a common precaution to prevent the freezing of water on the concrete surfaces in cold climates. However, this practice can result in salt scaling, one of the major durability issues in concrete.

Valenza II and Scherer (2007a) addressed a difference between salt scaling and usual freeze/thaw damage. They indicated that freeze/thaw damage caused by internal crystallization occurs with the stiffness and strength reduction while salt scaling does not have an influence on the mechanical properties of the concrete due to its superficial characteristics. They also mentioned that salt scaling is not visually pleasing due to the exposure of the coarse aggregate and the surface of a scaled specimen would be more sensitive to the penetration of fluids and detrimental ions like chloride which results in the corrosion of the reinforcing steel and concerns the durability of the concrete body.

Valenza II and Scherer (2007a) prepared a list of characteristics of scaling damage that every acceptable mechanism must take into consideration as follows:

> "1. Salt scaling consists of the progressive removal of small flakes or chips of binder.
> 2. A pessimum exists at a solute concentration of ~ 3%, independent of the solute used.
> 3. No scaling occurs when the pool of solution is missing from the concrete surface.

4. No damage occurs when the minimum temperature is held above −10°C; the amount of damage increases as the minimum temperature decreases below −10°C and with longer time at the minimum temperature.
5. Air entrainment improves salt scaling resistance.
6. The salt concentration of the pool on the surface is more important than the salt concentration in the pore solution.
7. Susceptibility to salt scaling is not correlated with susceptibility to internal frost action.
8. The strength of the surface governs the ability of a cementitious body to resist salt scaling."

2.2 Mechanism of salt scaling

Having the air void distance smaller than the critical spacing factor, strength loss is lowered or avoided by the entraining air (Powers and Helmuth, 1953; Helmuth, 1962; MacInnis and Beaudoin, 1968; Kobayashi et al., 1981; Pigeon et al., 1986). This issue beside the improvement of salt scaling resistance by the addition of air-entrainment led Lindmark (1998) and Sommer (1979) to suggest internal frost action as the reason of the salt scaling deterioration of concrete surfaces.

Several other mechanisms for salt scaling have been proposed as follows; but, only glue spall mechanism was found to explain all the characteristics of salt scaling damage mentioned in the previous section (Valenza II and Scherer, 2007b).

2.2.1 Internal crystallization

Internal crystallization has been considered as a reason for the salt scaling (Valenza II and Scherer, 2007b). The role of hydraulic pressure and crystallization pressure is discussed in the following sections.

2.2.1.1 Hydraulic pressure

When water freezes, it expands and forms into ice which has a volume 9% greater than water. The freezing of water and the ice formation initiates in the larger pores and voids then progresses into the smaller ones as the temperature further lowers which causes the water to be released from the capillaries as they freeze and creates the hydrostatic pressure (Boyd, 1995). Water will move from the freezing part of a permeable concrete sample by the formation of ice and the volume expansion (Valenza II and Scherer, 2007b) and the hydraulic pressure was addressed by Powers (1945) as the reason of this movement. Powers and Helmuth (1953) indicated that the detrimental expansion due to the internal crystallization can be lowered by minimizing the spacing between the air voids to 250-300 μm.

Boyd (1995) addressed the hydraulic pressure as dependent on the permeability, the freezing rate, and the distance between the capillary and the void edge. Valenza II and Scherer (2007b) indicated that the damage due to the internal crystallization is not related to hydraulic pressure and reported that this mechanism cannot explain the pessimum concentration or improved salt scaling performance when the top surface of the specimens are not covered with the saline solution.

2.2.1.2 Crystallization pressure

Some researchers observed damage in frozen permeable concretes having fluids with larger specific volume than the solid body (Beaudoin and MacInnis, 1974; Browne and Cady, 1975). It was observed by Valenza II and Scherer (2007b) that the melting point is lowered at the water solidification in a fine pore due to the higher surface-to-volume ratio

of the ice crystals, but the system energy is not reduced because there is a balance between the solidification energy and the energy of the interface of crystal and fluid. Scherer (1999) addressed a very high pressure required to push the crystals to be in contact with the pore wall as higher than the cement paste tensile strength. Some researchers addressed the crystal development around the wall placing force onto it (Corte, 1962; Uhlmann and Chalmers, 1964; Chernov et al., 1976).

Valenza II and Scherer (2007b) indicated that crystallization pressure creates detrimental expansion if entraining air is not used in the concrete mixtures and reported that this mechanism, like the hydraulic pressure, cannot explain the pessimum concentration or better salt scaling performance when the top surface of the specimens are not covered with the saline solution.

2.2.2 Role of salt

Several mechanisms have been proposed to study the effect of salt on the salt scaling resistance. These theories are explained in the following sections, but Valenza II and Scherer (2007b) indicated that none of them sufficiently explained all of the characteristics of salt scaling described earlier.

2.2.2.1 Thermal shock

The application of the de-icing salt solution results in a reduction in the melting point of ice. The heat required to melt the ice is taken from the concrete surface which results in a temperature gradient at the concrete surface and creates differential stress and strain (Valenza II and Scherer, 2007b). They mentioned that this minor thermal shock as the

result of the de-icing salt application on a frozen concrete surface layer cannot create detrimental stresses and concluded that this mechanism cannot explain the pessimum concentration and the salt scaling damage is not considerably related to this mechanism.

2.2.2.2 Precipitation and growth of salt

As ice is pure water, freezing a saline solution increases the salt concentration in the rest of the solution. In the salt scaling test, the concrete surface is exposed to a concentrated saline solution whose volume decreases by reducing the initial concentration of the saline solution. At -20 °C, the concentration of the solution increases to almost 22% by weight from an initial concentration of 3% (Valenza II and Scherer, 2007b). They concluded that in the temperature range used in the salt scaling test, damage from the salt precipitation does not happen and this mechanism also cannot explain the pessimum concentration.

2.2.2.3 Salt concentration in the pore solution

Generally, freezing damage occurs in concrete when the saturation degree exceeds the critical value (Verbeck and Klieger, 1957). Valenza II and Scherer (2007b) addressed an argument if the salt concentration in the pore solution can increase the saturation degree of the concrete surface. They also indicated that intermediate salt concentrations do not extremely increase the saturation degree although they severely scale the concrete surfaces.

2.2.2.3.1 Reduction in vapour pressure

Vapour pressure decreases with increasing salt concentration in solutions, causing higher degree of saturation. However, since the maximum scaling occurs at 3% concentration rather than higher concentrations, this theory has been dismissed by Valenza II and Scherer (2007b).

2.2.2.3.2 Osmotic pressure

The existence of different soluble materials in the pore water like alkalies, free lime and chloride increases concentration at the ice formation and its growth in the rest of capillary water. This difference in salt concentration creates the osmotic pressure and results in the movement of gel water to the capillaries where the ice will be formed (Boyd, 1995; Powers, 1956). It was reported by Valenza II and Scherer (2007b) that the osmotic pressure will not have a critical influence regarding the salt scaling.

2.2.3 Glue spalling

The glue spall mechanism was proposed by some researchers as the initial reason of the salt scaling damage (Valenza II, 2005; Valenza and Scherer, 2006). This method is used to decorate a glass surface with scallops (Gulati and Hagy, 1982 and 1973). This is done by sandblasting the surface of the glass, covering its surface with an epoxy at high temperature, and decreasing the temperature which causes the epoxy to shrink relatively more than the glass and creates tension in this layer, distributes the flaws in the glass layer where finally a thin glass scallop is removed (Valenza II and Scherer, 2007b).

Freezing a saline solution on the top surface of a concrete specimen creates an ice/concrete composite similar to the procedure mentioned above (Valenza II and Scherer, 2007b). By lowering the temperature of the ice/concrete composite under the melting point of the saline solution, the ice layer shrinks significantly more than the concrete layer beneath (5 times more) (Pounder, 1965; Gagnon and Jones, 2001; Scherer, 2004; Ciardullo et al., 2005) where the role of ice in the ice/concrete composite is the same as the epoxy in the epoxy/glass composite (Valenza II, 2005; Valenza and Scherer, 2006).

2.2.4 Diffusion theory

The free energy of gel water is similar to that of the capillary water at 0 °C which will be increased when the capillary water freezes, makes the gel water to move to capillaries resulting in the contraction of gel pores, preparing more water to freeze, and therefore more expansion (Boyd, 1995).

2.2.5 Supercooling

Boyd (1995) indicated that the freezing of the cement paste water occurs under the melting temperature and addressed the size of the capillary and the solution concentration as the parameters affect the pore water supercooling which will influence the ice distribution rate and therefore the hydraulic pressure progress. Radjy et al. (1972) observed the mechanical failure of the specimen caused by extreme supercooling which was followed by the freezing.

2.2.6 Litvan's model

Litvan (1976a and 1976b) observed the concrete drying and water travelling to the surface due to the vapour pressure difference between the supercooled water in the pores and the ice at the surface layer and addressed the possibility of mechanical failure due to the prevention of this movement as in low permeable surface layer.

2.2.7 Solar effects

The influence of the energy of sun on the freezing-and-thawing of the concrete surface was investigated by Moukwa and Adkins (1988). They observed a fast heating of the concrete surface by this energy and a two-directional freezing at its absence, from the top surface to the bottom and from the frozen concrete to the surface, which produces ice lenses, generates a weak layer, and scales the top layer.

2.3 Different test methods for salt scaling

The test methods used by several researchers regarding the salt scaling are as following:

- ASTM C 672/C 672M (2003), most of the experiments regarding the salt scaling were conducted in accordance to this method which will be described in detail in Chapter 3.
- The capillary suction of de-icing chemicals and freeze-thaw test (CDF), an upside down version of ASTM C 672 developed in Germany. In this method, before starting the test, the concrete slab with a thickness of 50 to 75 mm is maintained at 20 °C and 65% relative humidity until equilibrium and then is supported 3 mm above the bottom

of a stainless steel container having NaCl solution where 20 mm of insulation covers the slab top surface (bottom molded surface). The temperature in this test method varies twice a day between -20 °C and +20 °C and the mass loss is determined after 7, 14, 28, 42, and 56 freezing-and-thawing cycles (Valenza II and Scherer, 2007a).

- The Swedish Standard SS 13 72 44 (SIS, 1992), also known as Borås method, is a similar method to ASTM C 672 with two differences. First, the bottom and sides of the specimens are covered with almost 20 mm of insulation to assist unidirectional freezing. Second, to avoid the evaporation, a plastic foil sheet is placed 20 mm above the salt solution. In this test method, the thickness of the specimens is 50 mm, the depth of salt solution is 3 mm, the temperature is lowered to -15 to -20 °C during the first 12 hours followed by keeping the temperature for four hours at this minimum temperature followed by increasing the temperature to 15 to 25 °C during the next eight hours. The mass loss is determined after 7, 14, 28, 42, and 56 freezing-and-thawing cycles (Valenza II and Scherer, 2007a).
- BNQ NQ 2621-900 (2002) is another similar method to ASTM C 672 with some differences as follows. This method does not require brushing the surface after the bleeding; the concrete surface is covered with a polyethylene sheet right after finishing with a wooden trowel to minimize the evaporation. But its latest version requires a layer of dry non-weaved polypropylene geotextile with a thickness of 7 mm ± 1 mm at the bottom of the mold to provide some drainage which helps concrete mixtures sensitive to bleeding. The BNQ standard also requires a 13-day (after one day in the mold) moist curing at 23 ± 2 °C followed by a 14-day dry-curing at 15 to 30 °C and 50 ± 20 % relative humidity and finally a 7-day pre-saturation of the

concrete surface with the saline solution prior to the beginning of freezing-and-thawing cycles. ASTM C 672 requires Calcium chloride (4% $CaCl_2$) while the BNQ standard requires NaCl (with a concentration of 3.0 ± 0.1 %). The BNQ standard requires using a 3% NaCl solution to washing out the scaling residues from the specimen surface while most laboratories use pure water when tested according to ASTM C 672 (Bouzoubaa et al., 2008). In the BNQ standard, the scaling residues are collected and weighed at the end of 7, 21, 35, and 56 cycles of freezing-and-thawing but in the ASTM C 672, this process is done after every 5 cycles. The minimum surface area of the specimens in the BNQ standard is 0.05 m^2 versus 0.045 m^2 as required by ASTM C 672 and the visual rating in these two standards have different scales. And finally the BNQ standard requires stopping the freezing-and-thawing cycles where the cumulated mass loss exceeds 1.50 kg/m^2.

- The proposed replacement method for ASTM C 672, which is based on the BNQ standard and will be described in detail in Chapter 3. The major difference between the existing and proposed standard is the 7 day pre-saturation of the specimens. Bouzoubaa et al. (2008) found the ASTM C 672 method more severe in the laboratory than the field (as agreed to by Hooton and Boyd, 1997; Thomas, 1997; Langley and Leaman, 1998; Bleszynski et al., 2002; Marchand et al., 2005; Krishnan et al., 2006; Boyd and Hooton, 2007) and addressed the BNQ procedure as a better means to evaluate the salt scaling resistance of concrete incorporating the supplementary cementing materials. They reported greater scaling residue and higher visual rating of specimens tested in accordance with ASTM C 672 than the BNQ standard in concrete specimens made with fly ash and slag.

2.4 Parameters influencing salt scaling resistance

There are many parameters studied by several researchers which were found to have an effect on salt scaling resistance of a concrete surface. These parameters are discussed in detail in the following sections.

2.4.1 Supplementary cementing materials (SCMs)

Fly ash, silica fume, and ground granulated blast furnace slag are the common supplementary cementing materials used in concrete as Portland cement replacement. Generally with a proper design, replacing the Portland cement partially with supplementary cementing materials results in a comparable or improved mechanical properties and durability of concrete (Hassan et al., 2000; Blomberg, 2003; Nikam and Tambvekar, 2003; Bouzoubaa and Fournier, 2005; Bouzoubaa et al., 2008). Although using SCMs, especially fly ash and slag, in concrete usually improves the long-term strength, elastic modulus, permeability, and chemical resistance (Radlinski et al., 2008), these concretes may have a poor salt scaling resistance (Afrani and Rogers, 1994; Boyd, 1995; Pigeon et al., 1996; Bleszynski et al., 2002; Deja, 2003; Krishnan et al., 2006). In concrete mixtures made with supplementary cementing materials in the laboratory, some researchers noticed a thicker porous surface layer than the regular concrete which is more sensitive to the scaling while this layer was not observed in the field concretes which somehow can be the reason for their better scaling resistance (Pigeon and Marchand, 1996).

2.4.1.1 Silica fume

"Silica fume (or microsilica or condensed silica fume) is a by-product material of the manufacture of silicon or ferrosilicon alloys" (Valenza II and Scherer, 2007a). They indicated that the addition of silica fume to the concrete improves the early strength (age < 7 days) and the salt scaling resistance which could be due to the increased strength.

Some researchers indicated that the addition of silica fume to the concrete mixture having water-to-cement ratio $\geq$ 0.4 improves the salt scaling resistance if the compressive strength of the concrete mixture is higher than 40-45 MPa (Aitcin and Pigeon, 1986; Jacobsen et al. 1991) while a little influence on the salt scaling resistance was observed at lower w/c ratio (< 0.35) where the minimum compressive strength is 40-45 MPa (Sorensen, 1983; Gagne et al., 1991; Sellevold and Farstad, 1991). On the other hand, rapid and severe scaling was reported in silica fume concrete by extending the freezing-and-thawing cycles (Petersson, 1986; Sellevold and Farstad, 1991; Jacobsen, 1995).

2.4.1.2 Fly ash

Fly ash is a by-product of the coal-fired power industry having particles similar size to unhydrated cement grains. Concrete made with fly ash achieves higher strength at the later curing periods (1-2 months) (Valenza II and Scherer, 2007a).

Many researchers indicated that the salt scaling resistance decreases by the addition of fly ash (Gebler and Klieger, 1986; Johnston, 1987; Whiting, 1989; Bilodeau et al., 1991; Bilodeau and Malhotra, 1992). Part of its poor salt scaling resistance was found to be related to lower strength at short moist curing periods (3-28 days) (Whiting, 1989;

Bilodeau et al., 1991; Bilodeau and Malhotra, 1992). Better performance of class C fly ash was reported by Boyd (1995) than class F fly ash. Naik et al. (2005) indicated that postponing the salt scaling test to a later time after the concrete made with high-volume fly ash gets higher strength, would be useful.

Bilodeau et al. (1994) showed that addition of fly ash to the concrete significantly reduces its salt scaling resistance regardless of the type of fly ash. They used eight different fly ashes form U.S. sources at the replacement rate ranges from 55 to 60 % and observed the severe scaling (level 5) at the end of both 50 and 100 cycles with the exception of one mixture which showed moderate to severe scaling (level 4).

Bouzoubaa et al. (2008) compared the salt scaling resistance of laboratory specimens with sidewalk sections after four winters of outdoor exposure in accordance with both the ASTM C 672/ C 672M (2003) and BNQ NQ 2621-900 (2002). They compared the control mixture made with 25% blended silica fume cement (HSF, ~ 8% silica fume) and 75% regular GU Portland cement (ASTM type I) with mixtures containing one ASTM Class F fly ash (ASTM C 618, 2008) at 25% and 35% cement replacement. They kept the specimens in the mold for two days followed by 12 days moist-curing (instead of 13 days as required by ASTM C 672) and 3% NaCl solution was used in both the ASTM C 672 and the BNQ standard (instead of using 4% $CaCl_2$ in the ASTM C 672). They observed a poor salt scaling resistance (severe scaling, level 5) of the concrete mixtures incorporating fly ash in accordance with ASTM C 672 but a satisfactory resistance (level 2 for 35% fly ash and level 3 for 25%) according to BNQ (2002) in comparison with the control mixture (level 1 according to both ASTM C 672 and BNQ) which agrees with the findings of other researchers (Gebler and Klieger, 1986; Johnston, 1987; Whiting, 1989;

Bilodeau et al., 1991; Johnston, 1994; Hooton and Boyd, 1997; Talbot et al., 2000). At the end of 50 freezing-and-thawing cycles, Bouzoubaa et al. (2008) reported greater scaling residue and higher visual rating of specimens tested in accordance with ASTM C 672 than the BNQ standard in all control mixture and fly ash concretes. They indicated that all of the fly ash concrete specimens failed to meet the requirement of the Ministry of Transportation of Ontario, Canada (MTO) (having a mass loss smaller or equal to 0.8 kg/m^2) when tested in accordance with ASTM C 672. But when the BNQ standard was used, the requirement of having a maximum mass loss of 0.5 kg/m^2 was met. The results for sidewalks under field exposure will be discussed later.

Based on the results from the research done by Bilodeau and Malhotra (1992), it was found that most of the scaling occurred in the first 20 freezing-and-thawing cycles after which in the concrete mixtures with low 28-day strength, the rate of scaling was higher and the concrete mixtures without fly ash showed an acceptable salt scaling resistance without fast initial scaling (Valenza II and Scherer, 2007a).

On the other hand, Gebler and Klieger (1986) studied the effect of Class C and F fly ash (ten different fly ashes) on salt scaling resistance according to five different curing methods and found that the addition of fly ash (at 25% replacement) to the air-entrained concrete reduces salt scaling resistance regardless of the curing method or the type of fly ash. They also reported that the concrete mixtures made with class F fly ash showed slightly lower salt scaling resistance than the concrete mixtures containing class C fly ash only at lower temperature curing methods (4.4 °C) and the same resistance was observed at 23 °C curing methods.

2.4.1.3 Ground granulated blast furnace slag (GGBFS)

"Slag is a waste product of the pig iron production which must be cooled to produce granulated glass and after being cooled and highly grounded, it is called ground granulated blast furnace slag (GGBFS)" (Valenza II and Scherer, 2007a). They indicated that at younger ages (7-14 days), the concrete mixture incorporating the slag results in lower strength than the ordinary Portland concrete due to the slower hydration while higher strength of slag concrete is achieved after extended curing time (28 days).

A reduction in salt scaling resistance was reported by some researchers when the slag was added to the concrete (Bilodeau and Malhotra, 1993; Setzer, 1993; Copuroglu et al., 2004). Based on their results, Valenza II and Scherer (2007a) observed an increased scaling residue in the first ten freezing-and-thawing cycles which was reported to be related to the formation of a weak surface, not to the difference in the strength.

Afrani and Rogers (1994) addressed two potential reasons for the increased amount of scaling residue in the concretes made with slag; firstly, the early finishing before the bleeding has stopped due to the late set of slag concrete which may form a weak layer with a high w/c ratio so close to the surface, and secondly, not enough curing length due to a lower hydration heat of slag concrete at normal temperature.

Bouzoubaa et al. (2008) compared the salt scaling resistance of laboratory specimens with sidewalk sections after four winters of outdoor exposure in accordance with both the ASTM C 672/ C 672M (2003) and BNQ NQ 2621-900 (2002). When ASTM C 672 was used, they observed a better salt scaling resistance (lower scaling residue) of the concrete mixture incorporating slag at 25% cement replacement than the control mixture which met the requirement of MTO (having a mass loss smaller or equal to 0.8 kg/m^2) and

addressed the slightly higher air content of slag concrete as leading to its better salt scaling performance. On the other hand, the concrete mixture made with slag at 35% cement replacement was observed to have a lower salt scaling resistance (moderate to severe scaling, level 4) than the control mixture in accordance with ASTM C 672 and did not meet the MTO requirement but they showed a higher salt scaling resistance (level 1) when tested according to the BNQ standard (scaled significantly less) in comparison with ASTM C 672. They also reported that the concrete mixtures made with slag showed better salt scaling resistance than the fly ash concretes and addressed the higher air content and lower air-void spacing factors of the slag concretes as their better performance.

Afrani and Rogers (1994) studied the salt scaling resistance of six different concrete mixtures including 50% high-alkali Portland cement (HAPC) and 50% slag, 82% HAPC and 18% type F fly ash, 75% HAPC and 25% slag, 24% HAPC, 51% PSFC (blended cement having 7.5% silica fume and 25% slag), 100% low-alkali Portland cement, and 100% HAPC. They reported the greatest salt scaling resistance in 100% HAPC concrete mixture and the mixture made with 75% HAPC and 25% slag disregarding the curing method while the concrete mixture made with 50% high-alkali Portland cement and 50% slag was reported to have the least salt scaling resistance. They also reported the unexpected higher scaling residue (lower salt scaling resistance) of the concrete mixture made with the low-alkali cement cured under the standard curing method in comparison with the concrete mixture having the high-alkali cement.

2.4.1.4 Ternary blended cement

While using a single SCM in the concrete mixture shows poor salt scaling resistance, the ternary blends of cement with fly ash and silica fume were reported to improve the salt scaling resistance (Lilkov and Stoitchkov, 1996; Stoitchkov et al., 1996; Thomas et al., 1999; Olek et al., 2002; Blomberg, 2003; Nikam and Tambvekar, 2003; Bouzoubaâ et al., 2004; Radlinski et al., 2006).

On the other hand, Bouzoubaa et al. (2008) reported a poor salt scaling resistance of both ternary blended cements used in their study (fly ash-silica fume and slag-silica fume) in accordance with ASTM C 672. They observed a severe scaling (level 5) in the concrete mixture made with fly ash-silica fume ternary blended cement and a moderate to severe scaling (lever 4) in the concrete mixture made with slag-silica fume ternary blended cement and reported more scaling residue in the concrete made with fly ash-silica fume ternary blended cement than the concrete made with slag-silica fume ternary blended cement when tested according to ASTM C 672.

2.4.1.5 Replacement amount

Using large amounts of cement replacement with supplementary cementing materials (more than about 20% fly ash or 30% slag) throughout the laboratory tests was reported to show a poor slat scaling performance (Gebler and Klieger, 1986; Johnston, 1987; Whiting, 1989; Bilodeau et al., 1991; Johnston, 1994; Hooton and Boyd, 1997; Talbot et al., 2000). However, the other researchers reported a good salt scaling resistance of the concretes made with high amounts of these supplementary cementing materials in the

field (Hooton and Boyd, 1997; Thomas, 1997; Langley and Leaman, 1998; Marchand et al., 2005).

Neville (1996) reported the maximum effective replacement amount for fly ash, slag, and silica fume to be 30%, 50%, and 10%, respectively. The maximum amount of slag and fly ash replacement (when used individually) is limited to 25% and 10%, respectively by the Ontario Ministry of Transportation for the concrete mixtures exposed to the freezing-and-thawing, but in a concrete mixture made with both fly ash and slag, the maximum replacement amount of fly ash and the total slag and fly ash is limited to 10% and 25%, respectively (Afrani and Rogers, 1994). Most government agencies in Canada (municipalities, provincial departments of transportation) usually limit the maximum replacement amount of fly ash and slag to 20% or less in the concrete mixtures exposed to de-icing salts (Bouzoubaa et al., 2008).

Valenza II and Scherer (2007a) indicated that low replacement amount (< 10 %) of silica fume results in higher salt scaling resistance in a concrete mixture having a minimum compressive strength of 40 MPa. They also reported more scaling residue by using slag or increasing its replacement amount in the concrete based on the results from the experiments done by Bilodeau and Malhotra (1993) which is agreed by other researchers too (Afrani and Rogers, 1993; Setzer, 1993).

It was reported that by limiting the fly ash replacement amount to 30%, the salt scaling resistance will not be influenced significantly (Bilodeau et al., 1991; Afrani and Rogers, 1993). Naik et al. (2005) investigated the effect of different replacement amounts of class C fly ash in the laboratory concrete mixtures and reported no scaling or very slight scaling for up to 45% replacement, slight to moderate scaling for 50 to 60% replacement,

and severe scaling for 74% replacement. They also reported very slight scaling damage in the pavement sections having up to 50% class C fly ash. They addressed the possibility of making a concrete mixture by replacing up to 56% of cement with class C fly ash which will resist the salt scaling. They also reported slight to moderate scaling in the laboratory concrete mixture having 40% class F fly ash while only slight scaling was found in the field pavement sections.

Neuwald et al. (2003) reported more bleeding by increasing the fly ash replacement amount and the negative influence of high volume replacement of fly ash (> 30 %) on scaling was reported by the other researchers (Whiting, 1989; Afrani and Rogers, 1993). Bilodeau et al. (1994) reported the severe scaling (level 5) at the end of both 50 and 100 cycles in all concrete mixtures containing eight different fly ashes form U.S. sources at the replacement rate ranges from 55 to 60% with the exception of one mixture which showed moderate to severe scaling (level 4).

Using fly ash and slag or increasing their replacement amount in the concretes made with high-alkali Portland cement and these supplementary cementing materials resulted in lower salt scaling resistance (Afrani and Rogers, 1994). Boyd and Hooton (2007) also observed the negative effect of increasing the replacement amount of supplementary cementing materials on the salt scaling resistance. They have studied six different laboratory concrete mixtures made with slag (at 25, 35, and 50% cement replacement), fly ash (at 15% cement replacement), 25% slag combined with 10% fly ash, and 100% Portland cement. Except the concrete mixture containing 50% slag and the slag and fly ash ternary blend (25% slag and 10% fly ash), the other concrete mixtures were found within the required mass loss range by MTO. Radlinski et al. (2008) observed a reduction

in the salt scaling resistance by increasing the replacement amount of class C fly ash in the ternary concrete mixture due to the fact that ternary concrete mixture made with 20% class C fly ash and 5 or 7% silica fume showed higher scaling resistance than ternary concrete mixture made with 30% class C fly ash and 5 or 7% silica fume. They reported a very good salt scaling resistance of ternary concrete mixtures having 20% fly ash and 5 or 7% silica fume when testes at both the early and late ages while in the concrete mixtures made with 30% class C fly ash and 5% silica fume using the early age testing, a large amount of scaling residue (poor salt scaling resistance) was observed which was significantly decreased at the late age testing. They also reported a considerable scaling residue of the concrete mixture made with 30% class C fly ash and 7% silica fume when tested at the both ages.

Bilodeau et al. (1987) reported more scaling of the concrete mixtures made with different types of slag at 25% and 50% cement replacement in comparison with the concrete mixture made with Portland cement. In contrast, Fagerlund (1982) reported a significant improvement of salt scaling resistance when the replacement amount of slag was increased (to 65%) and observed a fair to good salt scaling resistance in the concrete mixtures made with slag at 65% cement replacement even without entraining air. However, Fagerlund (1982) used the Swedish method, an upside down version, covering the bottom surface of the samples with NaCl solution and conducting the test on the sawn surfaces of cylinders cured for seven months followed by one week drying at 50 °C before the test.

2.4.2 Mixture design factors

In this section the effect of mixture design factors on the salt scaling resistance is explained. These factors include the entraining air and spacing factor, compressive strength, water-to-cement ratio, and polypropylene fibre.

2.4.2.1 Entraining air and spacing factor

Several researchers reported the improvement of salt scaling resistance by the addition of air entrainment to the concrete (Verbeck and Klieger, 1957; Jackson, 1958; Sommer, 1979; Sorensen, 1983; Aitcin and Pigeon, 1986; Fournier et al., 1987; Siebel, 1989; Stark, 1989; Jacobsen et al., 1991; Sellevold and Farstad, 1991; Setzer, 1993; Marchand et al., 1995b). This improvement can be seen in Figure 2.1.

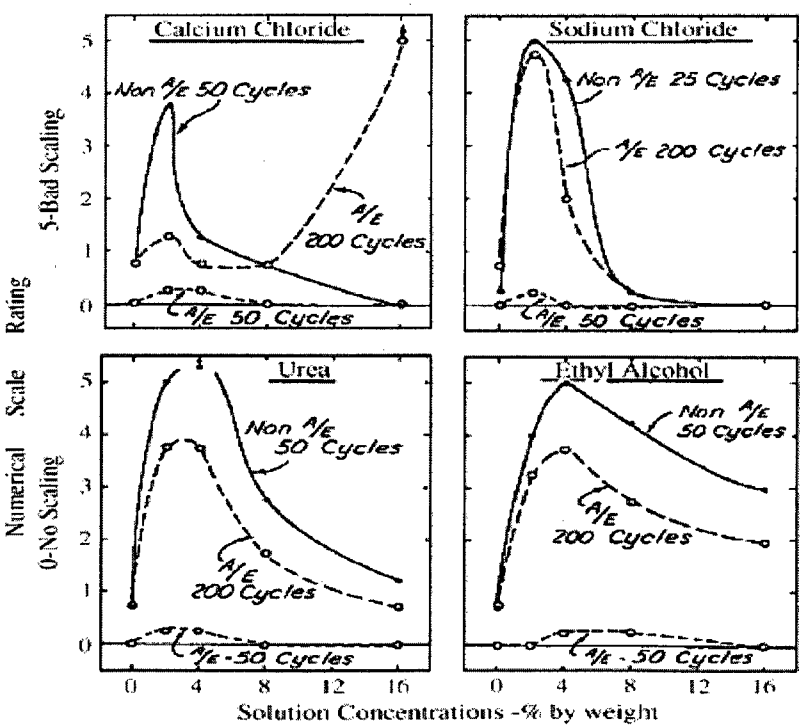

Figure 2.1 --- The relationship between solution concentration, entraining air, and scaling visual rating (Verbeck and Klieger, 1957).

Acording to Valenza II and Scherer (2007a), air entrainment improves the salt scaling resistance by reducing the bleeding and compressing the permeable sample which happens when the ice in the air voids absorbs pore liquid from the cement paste.

Deja (2003) showed that adding 5 to 6 % entraining air to the concrete mixture containing high volume granulated blast furnace slag (57%) increased its salt scaling resistance even at high w/c ratio (0.5). They observed a considerable scaling damage in non-air-entrained concrete mixture at early age (after 14 cycles) which was significantly decreased by the addition of the air-entraining admixture. They also showed that increasing the air content reduces the amount of scaling residue.

Several researchers reported the critical spacing factor of 250-300 μm and indicated that the concrete mixtures having a spacing factor smaller than the mentioned critical value show an acceptable salt scaling resistance regardless of the concrete type (Sommer, 1979; Fournier et al., 1987; Langlois et al., 1989; Siebel, 1989; Marchand et al., 1995a) while Bouzoubaa et al. (2008) indicated a maximum spacing factor of 200 μm for air-entrained concrete to prevent the salt scaling damage. Some researchers indicated that in the concrete mixture having an air void spacing greater than the critical limit, the mass loss is dependent on the spacing factor (Klieger, 1980; Siebel, 1989; Bordeleau et al., 1992). On the other hand, for internal frost action other researchers addressed a variable critical spacing factor dependent on the concrete type (Kobayashi et al., 1981; Pigeon et al., 1986; Foy et al. 1988).

Pigeon et al. (1986) as well as Backstrom et al. (1958) indicated that by increasing the volume of air entrainment in the concrete, the spacing factor is reduced which results in less bleeding (Bruere, 1958; Powers, 1968; Mindness and Young, 1981; Neville, 1996)

(Figure 2.2). Bruere (1958) indicated that air bubbles stick to the cement grains, making them float and result in less bleeding. He also observed that even without any air entrainment, some air-entraining admixtures result in less bleeding (Figure 2.2).

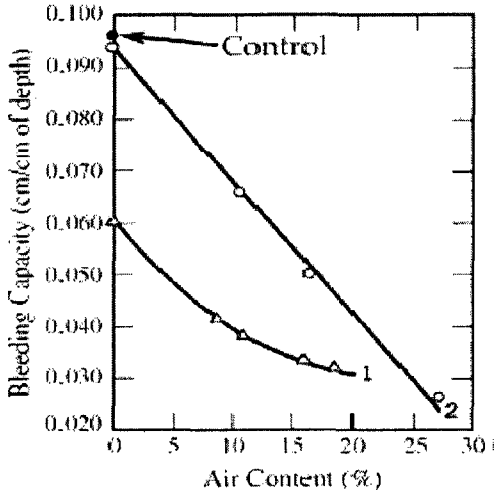

Figure 2.2 --- The relationship between air content and bleeding (Bruere, 1958). (Surface-active agents: 1-Sodium abietate, 0.05% by cement weight 2-Saponin, 0.10% by cement weight)

Siebel (1989) observed an increased scaling mass loss when the volume of air voids having a diameter smaller than 300 μm decreases and the critical volume was found to be around 1.5% (Siebel, 1989; Hammer and Sellevold, 1990; Setzer, 1993) (Figure 2.3).

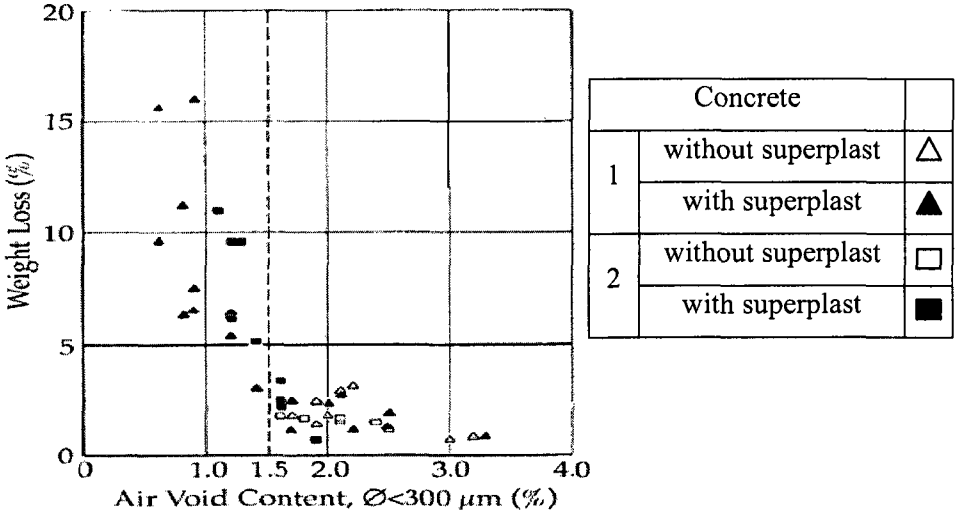

Figure 2.3 --- Air void content and weight loss relationship (Siebel, 1989).

2.4.2.2 Compressive strength

Bouzoubaa et al. (2008) reported a lower salt scaling resistance of fly ash concrete mixtures than control concrete and suggested the lower compressive strength of fly ash concrete at the beginning of the drying period (at the age of 14 days) as the reason of this poor performance. However, they also observed a lower salt scaling resistance in the concrete made with 35% slag as the cement replacement and the concrete mixtures incorporating fly ash-silica fume and slag-silica fume ternary blended cements is spite of having a higher 14-day compressive strength than the control mixture. Therefore, they indicated that the surface layer characteristics as well as the mechanical properties of the bulk concrete have an effect on the salt scaling resistance.

Valenza II and Scherer (2007a) indicated that in order for silica fume and fly ash concrete to have acceptable performance regarding salt scaling, a minimum compressive strength of 40-45 MPa is necessary, but not necessarily adequate. Bilodeau et al. (1991) as well as Bilodeau and Malhotra (1992) reported a significant reduction of the amount of scaling damage if w/c $\leq$ 0.5 or the 28-day compressive strength reaches 40 MPa. Naik et al. (2005) observed a lower salt scaling resistance in a concrete mixture having lower compressive strength than a higher-strength concrete.

2.4.2.3 Water to cement ratio (w/c)

Some researchers indicated that decreasing the water-to-cement ratio reduces the bleeding (Taylor, 1961; Powers, 1968; Neville, 1996) and increases the strength (Mindness and Young, 1981; Neville, 1996). Many researchers observed that reducing the water-to-cement ratio improves the salt scaling resistance (Jackson, 1958; Klieger and

Landgren, 1969; Sorensen, 1983; Petersson, 1986; Johnston, 1987; Bilodeau and Carette, 1989; Rose et al., 1989; Whiting, 1989; Whiting and Schmitt, 1989; Hammer and Sellevold, 1990; Bilodeau and Malhotra, 1992; Marchand et al., 1995a) and mentioned that in concretes with a w/c $\leq$ 0.3 there is no need for air entrainment regarding the salt scaling resistance (Foy et al., 1988; Gagne and Pigeon, 1990; Hammer and Sellevold, 1990; Jacobsen and Sellevold, 1993; Marchand et al., 1995a) due to a very little bleeding (Valenza II and Scherer, 2007a). Powers (1968) mentioned that a cement paste having w/c = 0.3 has a very low bleeding capacity like cement past having w/c $\approx$ 0.45 and 20% air.

2.4.2.4 Polypropylene fibre

Deja (2003) showed that adding the fibrillated polypropylene microfibres (length 19 mm) to the concrete mixtures containing high volume (57%) granulated blast furnace slag results in a significantly higher salt scaling resistance.

2.4.3 Fresh state

In this section the influence of fresh state on the salt scaling resistance is explained which include the bleeding and finishing.

2.4.3.1 Bleeding

It was observed by some researchers that bleeding or segregation varies the water-to-cement ratio and the density through the concrete thickness with the greatest density at the specimen's bottom (Taylor, 1961; Powers, 1968; Kreijger, 1984). Valenza II and

Scherer (2007a) indicated that the strength varies due to the density variation forming the weakest concrete at the top and suggested that the better salt scaling resistance of air-entrained concrete is due to the prevention of bleeding from reducing the surface strength. Afrani and Rogers (1993) as well as Janeva et al. (2002) conducted the salt scaling test on the bottom molded surface of the specimens and observed no scaling damage on the bottom surface while the top surface was damaged.

2.4.3.2 Finishing

Some researchers observed a greater salt scaling resistance when the surface of the specimens was brushed before the salt scaling test (Pigeon et al., 1987, Bilodeau and Carette, 1989; Bilodeau et al., 1991; Bilodeau and Malhotra, 1993).

On the other hand, Bouzoubaa et al. (2008) mentioned that by brushing the surface too early or much later (after the bleeding has stopped), the surface air-void network may be damaged which results in lower salt scaling resistance. After comparing the salt scaling results of ASTM C 672 and BNQ standards, they suggested that skipping the brushing (not touching the surface) improves the salt scaling resistance mainly with a slight bleeding. Bilodeau et al. (1994) also indicated that finishing may change the air-void system, making it insufficient to resist the freezing-and-thawing damage, resulting in poor scaling performance in concrete mixtures containing high volume of fly ash (at 55 to 60 % cement replacement).

Bouzoubaa et al. (2008) indicated that the field-type finishing of the concrete surface resulted in more scaling residue than the laboratory-type finishing. They addressed a

rougher surface of the field-finished samples having greater paste content as the reason of their worse performance.

Boyd and Hooton (2007) observed a slight influence of different finishing time (early finishing and normal finishing), related to bleeding, on the salt scaling resistance of concrete in the field. On the other hand, Taylor et al. (2004) reported the benefit of the early finishing in fly ash and slag concrete mixtures and the late finishing in the Portland cement concrete mixtures. They made three concrete mixtures with 100% type I Portland cement, grade 100 slag at 50% cement replacement, and class C fly ash at 25% cement replacement and did the finishing of the specimens at three different times including very soon after filling the molds, after bleeding stopped, and near initial setting. They observed the lowest salt scaling resistance at early finishing in the Portland cement concrete mixture which was improved by postponing the finishing while the opposite result was observed in fly ash and slag concrete mixtures in which the best salt scaling resistance was observed at the earlier finishing times.

2.4.4 Curing

The importance of initial curing of concrete was reported by many researchers, especially in the concrete mixtures made with slow reaction materials such as fly ash and slag (Khan and Ayers, 1993; Shafiq and Cabrera, 2004). Increased salt scaling resistance of a Class F fly ash concrete mixture was observed when the moisture equilibrium was achieved before the salt scaling test (Waktola et al., 2005) which is verified by Krishnan et al. (2006) who also addressed the advantage of extended curing in fly ash and slag

concretes. In contrast, Bilodeau et al. (1991) reported more scaling as a result of extended moist curing.

Some researchers reported a lower salt scaling resistance by curing at high temperature for a short time (~24 hours) (Langlois et al., 1989; Jacobsen and Sellevold, 1993). Curing temperature has a negative influence on the durability (Jacobsen and Sellevold, 1993). The 28-day strength is reduced 10% by one-day curing at 38 °C (Richardson, 1991) and 25% by 18-hour curing at 70 °C (Langlois et al., 1989). Therefore, Valenza II and Scherer (2007a) concluded that curing at high temperatures reduces the strength and makes the specimens more sensitive to the salt scaling. Langlois et al. (1989) as well as Gagne et al. (1991) addressed the increased salt scaling resistance by extended moist curing.

Drying ordinary concrete at 45 °C (Sorensen, 1983) or 100 °C (Bilodeau and Carette, 1989) results in more sensitivity to the salt scaling while drying at room temperature (~23 °C) has a slight influence on the salt scaling resistance (Verbeck and Klieger, 1957; Whiting, 1989; Bilodeau et al., 1991). Valenza II and Scherer (2007a) indicated that drying at high temperature or at very low relative humidity (<30%) increases the sensitivity to the salt scaling by forming a weak surface of the concrete. It was observed that in the concrete mixtures made with silica fume, drying at 45 °C does not affect the durability (Sorensen, 1993) while a reduction in durability was observed by drying these mixtures at 100 °C (Bilodeau and Carette, 1989).

Bilodeau et al. (1991, 1998) indicated that in fly ash concrete, extending the moist curing period in the laboratory (from 28 to 91 days) decreased the salt scaling resistance while an improvement in the salt scaling resistance was observed by decreasing the

drying period before the salt scaling test. Afrani and Rogers (1993) showed that covering the specimens right after the casting with a plastic sheet until the demolding time (24 hours later) reduced the salt scaling resistance, while Neuwald et al. (2003) reported a significant increased salt scaling resistance by allowing evaporation after the casting and then sealing after a short time (6 hours). This technique would permit bleed water to evaporate, but still promote reasonable curing.

Radlinski et al. (2008) studied the effect of four different curing methods on salt scaling resistance which included air drying (storage at 23 °C and 50% relative humidity), three and seven days curing under wet burlap and covered with plastic sheet, and seven days covered with curing compound (applied on all surfaces except the bottom right after demolding which was removed by a wire brush after seven days). They reported lower sensitivity of the mixtures made with 20% class C fly ash to the initial curing conditions in comparison with the mixtures having 30% fly ash. They also observed that air drying (lack of curing) at the early age may decrease the salt scaling resistance at the late age and found that increasing the curing duration of the wet burlap from three to seven days resulted in slightly better salt scaling resistance at the early age which disappeared at the late age. And finally they observed that using the curing compound resulted in the best salt scaling resistance at the early age and a very good resistance at the late age which is in line with the findings by Krishnan et al. (2006). Radlinski et al. (2008) suggested that a thin layer of curing compound prevents the salt and moisture; reduces the saturation degree of the pore system in the very top layer, and results in the good performance at the early age in these specimens.

Copuroglu et al. (2004) studied the effect of eleven curing methods on salt scaling resistance of slag mortars including:

- Five weeks curing with 3% NaCl at 20 °C
- Five weeks immersed in tap water and sealed in a plastic bag at 20 °C
- Six days water curing followed by three weeks placed in desiccators having 0% relative humidity and 0% CO_2 at 20 °C before 3% NaCl saturation period (extremely dried, no carbonation)
- Six days tap water curing followed by three weeks placed in desiccators having 50% relative humidity and 28% CO_2 at 20 °C followed by one week 3% NaCl saturation (dried, no carbonation)
- Six days water curing followed by three weeks placing in the laboratory having 50% relative humidity at 20 °C before 3% NaCl saturation period (dried, carbonated)
- Six days water curing followed by three weeks placing in a container having 3% CO_2 and 50% relative humidity at 20 °C before 3% NaCl saturation period (dried, rapid carbonation)
- Five weeks curing by demineralised water at 20 °C (demi water)
- Five weeks curing by tap water at 20 °C (plain water)
- Five weeks curing by saturated $CaSO_4$ solution at 20 °C
- Five weeks curing by saturated lime solution at 20 °C
- Curing with one molar NaOH solution at 20 °C to maintain the pH above 13 and increase the activation rate of slag.

Copuroglu et al. (2004) addressed the critical influence of carbonation on salt scaling resistance of slag mortars at a fixed relative humidity. They also compared the non-

carbonation curing methods (extremely dried and dried) and found that by increasing the drying rate, the salt scaling resistance decreased. They mentioned that the carbonation has more influence on the salt scaling resistance of slag mortars than the drying. They finally addressed the limewater curing to have the best salt scaling resistance and the carbonated curing methods (dried-carbonated and dried-rapid carbonation) to have the worst salt scaling resistance.

Afrani and Rogers (1994) studied the effect of five different curing methods on salt scaling resistance of six concrete mixtures made with low and high alkali Portland cement, slag, fly ash, and silica fume. The curing regimes in their study included:

- 14-day moist curing at 23 °C followed by 14 days curing in the laboratory air
- 14-day outdoor curing under wet burlap covered with polyethylene followed by 14 days curing in the laboratory air
- Covering the concrete surface with a resin-based curing compound, Sealtight 1220®, and 28 days curing in the laboratory air
- Finishing the concrete surface by using a magnesium hand float, brushing it with a broom, covering it with the same curing compound, 14 days outdoor curing followed by 14 days curing in the laboratory air
- Finishing like the previous method followed by 14 days outdoor curing under wet burlap covered with polyethylene followed by 14 days curing in the laboratory air

Afrani and Rogers (1994) addressed the outdoor curing under wet burlap covered with polyethylene as the best curing method used in their study with the exception of one mixture made with 50% high-alkali Portland cement (HAPC) and 50% slag where the moist curing showed a better salt scaling resistance. They reported a good performance of

the curing compound regarding the salt scaling at the early cycles and a rapid scaling after five cycles in the weaker concrete mixtures and also the negative effect of the laboratory curing in ambient condition.

Jonsson and Olek (2004) studied the influence of high curing temperatures on salt scaling resistance of four non-air-entrained high-strength concretes prepared at different temperatures (10 to 35 °C). They placed the concrete into a big polystyrene container and recorded the concrete temperature continuously observing the maximum temperature ranged from 59 to 69 °C. They reported a significantly lower salt scaling resistance of the temperature-match-cure (TMC) specimens (7 to 63 times less) than the normal specimens cured at ambient temperature and indicated that higher maximum curing temperature resulted in more scaling.

Gebler and Klieger (1986) also studied the effect of five different curing methods on salt scaling resistance in the fly ash concrete mixtures (at 25% cement replacement) including:

- One day storage in the molds, covering with a damp burlap at 23 ± 1.7 °C followed by 13 days moist curing at 23 ± 1.7 °C followed by 14 days air curing at 23 ± 1.7 °C and 50 ± 5% relative humidity (moist curing at 23 °C)
- One day storage in the molds at 23 ± 1.7 °C without a protection followed by air curing at 23 ± 1.7 °C and 50 ± 5 % relative humidity until the age of 28 days (air curing at 23 °C)
- Application of a curing compound on top surface of the specimen followed by a storage at 23 ± 1.7 °C and 50 ± 5 % relative humidity until the age of 28 days (curing compound at 23 °C)

- One day storage in the molds, covering with a damp burlap at 4.4 ± 1.7 °C followed by 13 days moist curing at 4.4 ± 1.7 °C followed by 14 days air curing at 4.4 ± 1.7 °C and 95 ± 5% relative humidity (moist curing at 4.4 °C)
- Application of a curing compound on top surface of the specimen followed by a storage at 4.4 ± 1.7 °C and 95 ± 5 % relative humidity until the age of 28 days (curing compound at 4.4 °C)

Gebler and Klieger (1986) indicated that in concrete mixtures containing 25% fly ash, a slightly better salt scaling resistance of the method using the curing compound at 23 °C was observed than the air curing and the moist curing at this temperature while in the concrete mixtures without fly ash, all the curing methods at 23 °C showed almost the same salt scaling resistance. On the other hand, they reported a better salt scaling resistance of low-temperature moist curing (at 4.4 °C) in both concrete mixtures with and without fly ash than the high-temperature moist curing (at 23 °C) while almost the same salt scaling resistance was observed in the method using the curing compound at both low and high curing temperatures.

It has been reported that using the sealers or curing compounds (surface treatment) postpones the surface scaling (Afrani and Rogers, 1993; Sedran et al., 1993; Setzer, 1993; Cleland and Basheer, 2002) but has no influence on the salt scaling resistance at the end (Perenchio, 1988; Setzer, 1993; Cleland and Basheer, 2002). No effect on salt scaling resistance was observed by using the sealers on bridge decks (Perenchio, 1988).

Sedran et al. (1993) investigated three sealers including a silane, an oligometric siloxane and a polymeric siloxane while Afrani and Rogers (1993) used sealtight® 1220, a resin-based curing compound which all postponed the moisture penetration throughout

the salt scaling test. Klieger and Perenchio (1963) used a similar silicone surface treatment and found that an untreated specimen achieves similar moisture content to a treated specimen after being submerged for one day while this small difference does not exist after three days.

Cleland and Basheer (2002) indicated that the scaling damage is postponed significantly when the depth of the treated surface increases due to the removal of this surface throughout the initial cycles. Valenza II and Scherer (2007a) mentioned that in treated specimens, most of the cracks form in places with little or no treatment, and develop to a depth under the treated layer, and therefore, result in the removal of this layer. Boyd (1995) indicated that the pores in the surface of fresh concrete may be blocked when the curing compound is applied which stops the moisture penetration and leads to a "false" good frost resistance.

On the other hand, Pigeon et al. (1987) reported an improved salt scaling resistance in concrete made with silica fume by the application of a curing compound. In their study, the curing compound was applied after one day in the mold followed by a 6-day curing followed by brushing the surface with a metallic mechanized brush to take away as much as the weak layer having the curing compound prior to the test. Other researchers also reported the better performance of some curing compounds in comparison with long moist curing periods (Klieger and Gebler, 1987; Langlois et al., 1989; Marchand et al., 1992). Boyd and Hooton (2007) reported a better salt scaling resistance of the specimens treated with the curing compound in comparison with the specimens covered under burlap and plastic sheet.

2.4.5 Age

In this section the effect of age on the salt scaling resistance is explained which include the late exposure and maturity.

2.4.5.1 Late exposure to the freezing-and-thawing cycles

Radlinski et al. (2008) studied the effect of the exposure time to the freezing-and-thawing cycles on salt scaling resistance initiating the test at the early age (14, 17, or 21 days based on the initial curing method) or at the late age (90 days). For the scaling testing at early age, the initial curing was followed by the 14-day drying period (23 °C and 50% relative humidity) prior to the test while for the test at late age, the drying period was extended until the age of 69 days after the initial curing followed by 7 days re-saturation in a moist room having 100% relative humidity followed by 14 days air-drying at 23 °C and 50% relative humidity before the scaling test. They indicated that the late exposure did not considerably increase the salt scaling resistance and addressed the different scaling modes regarding the exposure time as the possible reason. They observed the uniform surface scaling (thin flakes of paste) due to the paste failure in specimens tested at the early age while popouts (large and thick chips) was observed in the samples tested at the late age due to the failure at the paste-aggregate interface.

2.4.5.2 Maturity

Bouzoubaa et al. (2008) prepared two cores from each slab cast in the field prior to the starting of the freezing-and-thawing cycles in the field (at the age of about 180 days) and conducted the salt scaling test on them according to ASTM C 672 in the laboratory to

study the effect of the maturity on the salt scaling resistance. They compared the scaling residue and visual rating of these cores with the cores subjected to the freezing-and-thawing cycles at the age of 28 days and found that this extended curing under field conditions (wetting and drying) considerably increased the salt scaling resistance of all concrete mixtures expect one. Bilodeau et al. (1991, 1998) indicated that in fly ash concretes, extending the moist curing period from 28 to 91 days in the laboratory decreased the salt scaling resistance while shorter drying periods before starting of the freezing-and-thawing cycles increased the salt scaling resistance. Bouzoubaa et al. (2008) suggested that the results of the preceding authors are due to the extended curing time and lack of drying before the test as the possible reasons for the better salt scaling resistance of the cores tested at the age of 180. Bouzoubaa et al. (2008) indicated that the extended maturing period (180 days), consisting of several wetting and drying cycles which cannot easily be simulated in the laboratory, increased the salt scaling resistance.

2.4.6 Environmental factors

In this section the effect of environmental factors on the salt scaling resistance is explained. These factors include the saline solution, minimum temperature, freezing rate, and osmotic pressure.

2.4.6.1 Saline solution

Several researchers observed the maximum salt scaling damage at a moderate salt concentration in the saline solution, 3% by weight (pessimum of solute concentration) (Arnfelt, 1943; Verbeck and Klieger, 1957; Petersson, 1986; Sellevold and Farstad, 1991;

Studer, 1993; Lindmark, 1998; Marchand et al., 1999) (Figure 2.1). Arnfelt (1943) as well as Verbeck and Klieger (1957) observed that the type of the saline solution does not have any effect on this pessimum (Figure 2.1). Verbeck and Klieger (1957) used calcium chloride, sodium chloride, urea, and ethyl alcohol in their study while Arnfelt (1943) used barium chloride, potassium ferrocyanide, and potassium ferricyanide. It was also observed that by keeping the top surface of the specimens free of saline solution, the salt scaling will not occur (Verbeck and Klieger, 1957; Sellevold and Farstad, 1991; Studer, 1993).

McDonald and Perenchio (1997) studied the effect of salt type on salt scaling resistance of the concrete. They used 4% salt solutions including 80% rock salt with 20% calcium chloride ($CaCl_2$), 100% rock salt, proprietary salt, 69% rock salt with 30% KCl and 1% $CaCl_2$, 50% rock salt with 50% KCl, and the deionized water (for the control mixture). They observed more scaling damage in the concrete mixtures with higher strength (lower water-to-cement ratio) than the mixtures having lower strength when the surface of the specimens was ponded by the proprietary salt. In two out of three concrete mixtures made in their study, the salt scaling damage due to the ponding with the proprietary salt was considerably lower than the other saline solutions except the pure deionized water which could be due the large amount of magnesium of the proprietary salt. They also indicated that ponding the specimens with the salt solutions having KCl resulted in the worst salt scaling damage.

2.4.6.2 Minimum temperature

Several researchers reported that increasing the minimum temperature of the thermal cycles decreases the scaling damage (Hammer and Sellevold, 1990; Sellevold and Farstad, 1991; Studer, 1993; Lindmark, 1998). It was observed that the scaling damage reduces more by increasing the minimum temperature from -18 °C to -16 °C than increasing it from -13 °C to -11 °C (Studer, 1993). The other researchers indicated that keeping the minimum temperature above -10 °C results in no scaling damage (Sellevold and Farstad, 1991; Marchand et al., 1995b) while more scaling damage was observed in the specimens kept for longer period at the minimum temperature (-10 °C) (Sellevold and Farstad, 1991; Jacobsen, 1995).

2.4.6.3 Freezing rate

Studer (1993) as well as Marchand et al. (1995b) indicated that the rate of freezing does not have a huge effect on the scaling damage.

2.4.6.4 Osmotic pressure

Cordon (1966) addressed the osmotic pressure as one of the major reasons of the salt scaling. Bouzoubaa et al. (2008) mentioned that the one-week pre-saturation period as used in the BNQ standard may balance the ions between the saline solution on top surface and the top layers' voids, decrease the osmotic pressure and result in less salt scaling.

2.4.7 Field exposure

Bouzoubaa et al. (2008) visually evaluated sidewalks after four winters and observed a good performance in the control and slag concretes (25% and 35% cement replacement with slag and the concrete made with slag-silica fume ternary blended cement), acceptable performance in the fly ash concretes (at 25% and 35% cement replacement), and poor performance in the concrete made with fly ash-silica fume ternary blended cement. The salt scaling test was conducted according to ASTM C 672 and the visual ratings are presented in Table 2.1.

Table 2.1 --- Visual rating of sidewalk sections after four winters (Bouzoubaa et al., 2008).

Concrete type	Visual rating
Control	0
35% fly ash	2-3
35% slag	0-1
25% fly ash	3
25% slag	1-2
Fly ash-silica fume ternary blended cement	>4
Slag-silica fume ternary blended cement	0-1

As it can be seen in Table 2.1, the field evaluation by Bouzoubaa et al. (2008) showed the greater salt scaling resistance of the sidewalks made with slag concretes in comparison with the ones made with fly ash concretes which agrees with their laboratory results. Also an acceptable salt scaling resistance of all sidewalks except the one made with fly ash-silica fume ternary blended cement was observed by them in accordance with ASTM C 672 (having visual ratings of 0 to 3). After comparing the salt scaling

resistance of the field and laboratory specimens, they found ASTM C 672 to be a severe salt scaling test method while the BNQ standard was found to be a better way to describe the salt scaling resistance of the concrete mixtures incorporating the supplementary cementing materials.

Boyd and Hooton (2007) indicated that the specimens subject to field exposure (removed from the field after 127 days and tested at the laboratory) showed a significantly increased salt scaling resistance and met the requirement of MTO except the mixture containing 50% slag. They addressed the severity of the laboratory testing method in comparison with the field condition. Some visible scaling was observed in only one concrete mixture (containing 50% slag under moist curing with burlap and plastic) after a ten-year exposure while slight scaling was found in the other specimens.

Chapter 3

Experimental program

3.1 General

The experimental program was planned to study the salt scaling resistance of concrete surfaces exposed to freezing-and-thawing cycles in the presence of de-icing chemicals. In this study, ASTM C 672/C 672M (2003) was compared to its proposed replacement method (WK 9367) which is based on the method developed by Bureau de Normalisation du Quebec (BNQ). As specified in the proposed replacement method, the current ASTM C 672 method has been reported to be more severe than field experience i.e. concrete that fails the test has been observed to perform satisfactorily in the field. The particular concerns from the point of view of the proposed replacement method are severity of osmotic effects in samples not pre-saturated with salt solution and the significant influence of finishing activities and curing.

Besides the two test methods, other variables were investigated. Two additional types of curing, a specific curing compound and a three-day lime-water curing, were studied. Finally the concretes with cement replacements of 25% fly ash and 35% slag were compared with the plain concrete regarding the salt scaling resistance.

The concrete mixing, casting of the samples, slump test, air content measurement and compression tests were conducted in the structural laboratory of Concordia University located in Hall building's basement while curing and the freezing-and-thawing cycles of

the samples were carried out in the building materials laboratory located on the 15th floor of E.V. building.

3.2 Materials

In this section, all the materials used in this research including aggregates, cement, supplementary cementing materials (fly ash and slag), and chemical admixtures are described in detail.

3.2.1 Aggregates

The coarse and fine aggregates used in concrete mixtures in this experimental research were obtained from St. Lawrence Cement from a local source near Montreal, Quebec. The tests conducted on the fine and coarse aggregates included sieve analysis, specific gravity, absorption, bulk density, and moisture content which will be explained in more detail later.

3.2.2 Cementitous materials

In this study, Portland cement, fly ash, and slag were used as cementitous materials which are described in detail as follows.

One type of Portland cement, General Use Hydraulic cement (Type GU), was used in this experimental research. The cement was provided by St. Lawrence Cement, Joliette, Quebec, Canada. The chemical analysis and physical tests are given in Tables 3.1 and 3.2 respectively.

Table 3.1 --- The chemical analysis of the cement used in the concrete mixtures (%).

Chemical composition		Mineralogical composition	
Alkalies (Na_2O equi.)	0.88	C_3S	54
Loss on ignition	2.2	C_2S	18
Insoluble residue	0.7	C_3A	7
Free lime	0.8	C_4AF	8
SiO_2	20.6		
Al_2O_3	4.5		
Fe_2O_3	2.7		
CaO	62.7		
MgO	2.0		
SO_3	3.6		

Table 3.2 --- The physical tests of the cement used in the concrete mixtures.

Fineness: Blaine Residue 45 μ	392 m^2/kg 5 %
Autoclave expansion (A3004-B5)	0.004 %
Expansion in water (A3004-C5)	0.009 %
Setting time: Initial Final	 180 min. 240 min.
Air content	7.4 %
False Set	67 %
Compressive strength 3 days 7 days 28 days	 27.0 MPa 31.4 MPa 39.7 MPa

The fly ash used in this research was type F which was obtained from a source in Quebec. Fly ash was used in concrete mixtures at 25% replacement of cement.

The slag used in this research was Grade 100 which was obtained from Hamilton, Ontario. Slag was used in concrete mixtures at 35% replacement of cement.

3.2.3 Chemical admixtures

All of the following chemical admixtures used in this research are products of Master builders, BASF chemical company. All the information about these products was derived from Master builders' product information obtained from their website. The additional information of these chemical admixtures including their features, benefits, performance characteristics, applications, guidelines for use, storage and handling, packaging, and so on are available in Appendix A.

To achieve adequate cement dispersion at the relatively low water to cement ratio used, PolyHeed® 997 - a mid-range water-reducing admixture meeting ASTM C 494/C 494M (2008) Type A and F - was used in all concrete mixtures. This chemical admixture was added to the water used for concrete mixing. In all six concrete mixtures in this study, 35 mL of this admixture was used for 30 litres of concrete (292 mL/100kg of total cementing material).

To keep the slump within the required range (75 ± 15 mm) as specified in ASTM C 672, a high-range water-reducing admixture, Glenium® 7500, was also used in all concrete mixtures. This admixture was added to the concrete mixtures gradually. Usually half of the predicted amount was added to the water which was used for making the concrete and the other half was added during the three minutes rest time after the first

three minutes mixing. After mixing for the additional two minutes, the slump was measured and if the slump was not within the range, an additional amount of admixture was added followed by mixing for one minute and performing the slump test again. This procedure was repeated until the slump fell within the required range. The amounts of used admixture for each concrete mixture will be presented in section 3.4.

To meet the required air content (6 ± 1 %) as specified in ASTM C 672, an air-entraining admixture meeting ASTM C 260 (2006), MB-AE™ 90, was used in all six concrete mixtures. This chemical admixture was added to the fine aggregate. The amounts of this admixture used for each concrete mixture will be presented in section 3.4.

3.2.4 Supply water

All the water used in concrete mixing and preparation of calcium chloride solution was the city of Montreal tap water.

3.2.5 Curing compound

An evaporation reducer, Confilm®, was used in this research to see the effects of a curing compound on the salt scaling resistance of concrete surfaces. First one part of Confilm® evaporation reducer was mixed with nine parts of water to produce ten parts of sprayable solution as instructed by the manufacturer. Then based on the surface area of the scaling specimens, the mass of Confilm® evaporation reducer needed for each scaling specimen was determined. Since the product guidelines of Confilm® specified that 37.9 litres of solution should cover 186 to 372 m^2 (101.9 to 203.7 millilitres per one m^2) of the surface area, for each scaling specimen 8 millilitres per 0.0554 m^2 (135.84 millilitres per

one m^2) of the solution were sprayed on the surface, after the scaling specimens were prepared, the final finishing was done and the bleeding stopped.

3.2.6 Calcium chloride solution

As specified in ASTM C 672 and its proposed replacement method, the saline solution used for the salt scaling test, the pre-saturation period in the proposed replacement method, and rinsing the surfaces of the scaling specimens after each five cycles was a mixture of calcium chloride and water, having a concentration such that each 100 mL of solution contains 4 g of anhydrous calcium chloride. In this research, ACS certified calcium chloride dehydrate ($CaCl_2.2H_2O$) was used with a molecular weight of 147.01 g. Therefore, its anhydrous equivalent was calculated and used in the tests. Instead of 4 g of anhydrous calcium chloride, 5.297 g of dihydrate were used in each 100 mL of solution.

3.3 Tests conducted on the aggregates

In this study, the tests conducted on the aggregates included sieve analysis, specific gravity, absorption, and bulk density which will be explained in more detail in the following sections. Also the moisture content of the aggregates was measured for each concrete mixture which will be described in more detail in the concrete mixing and casting section.

3.3.1 Sieve analyses

Sieve analyses were conducted on both the fine and coarse aggregates used in this research in accordance with ASTM C 136 (2006). The fineness modulus of the fine

aggregate was calculated to be 2.64 which is within the accepted range of between 2.3 and 3.1. The result for sieve analysis of the fine aggregate is presented in Figure 3.1. The minimum and maximum limits presented in Figure 3.1 are according to ASTM C 33 (2007). As it is observed in Figure 3.1, the passed percentage of the fine aggregate used in this study is within the minimum and maximum limits in accordance with ASTM C 33 (2007) except for the sieve # 4 (4.76 mm) which had 93.20% passed, a little lower than the minimum limit (95%).

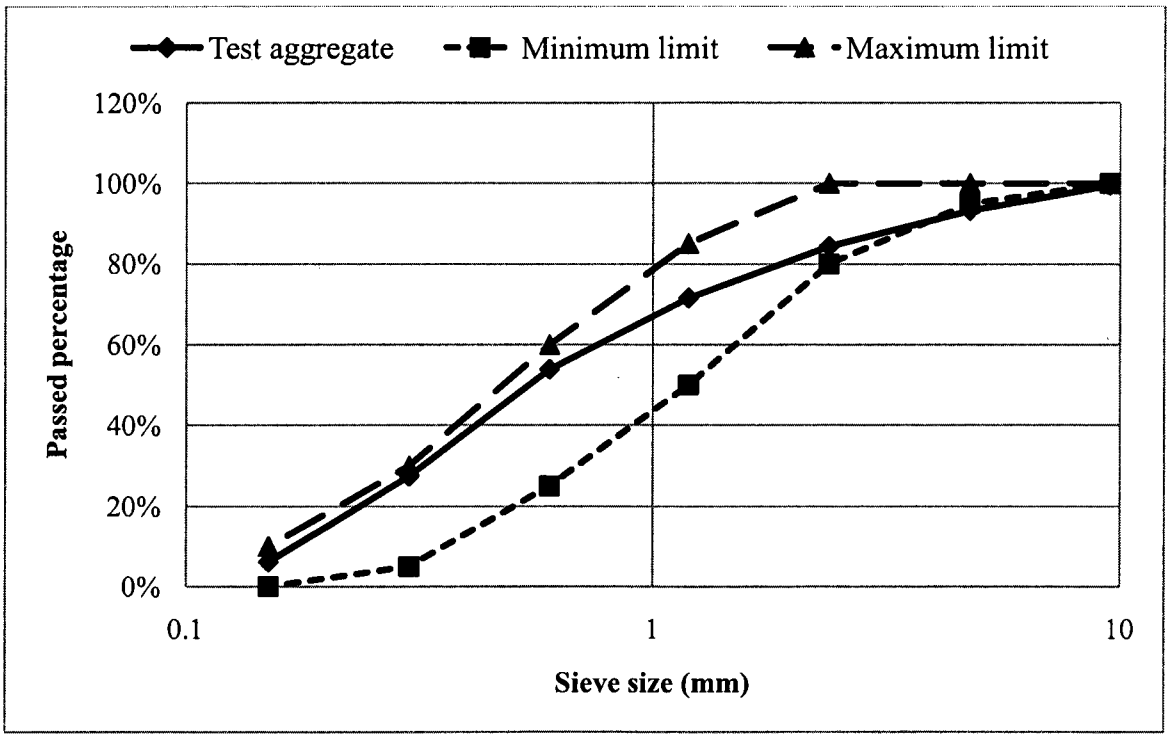

Figure 3.1 --- Sieve analysis of the fine aggregate used in this study.

The nominal maximum aggregate size of the coarse aggregate was determined to be 10 mm and the result for sieve analysis of the coarse aggregate is presented in Figure 3.2. The minimum and maximum limits presented in Figure 3.2 are according to ASTM C 33

(2007). As it is observed in Figure 3.2, the passed percentage of the coarse aggregate used in this study is within the minimum and maximum limits corresponding to size number 8 in accordance with ASTM C 33 (2007).

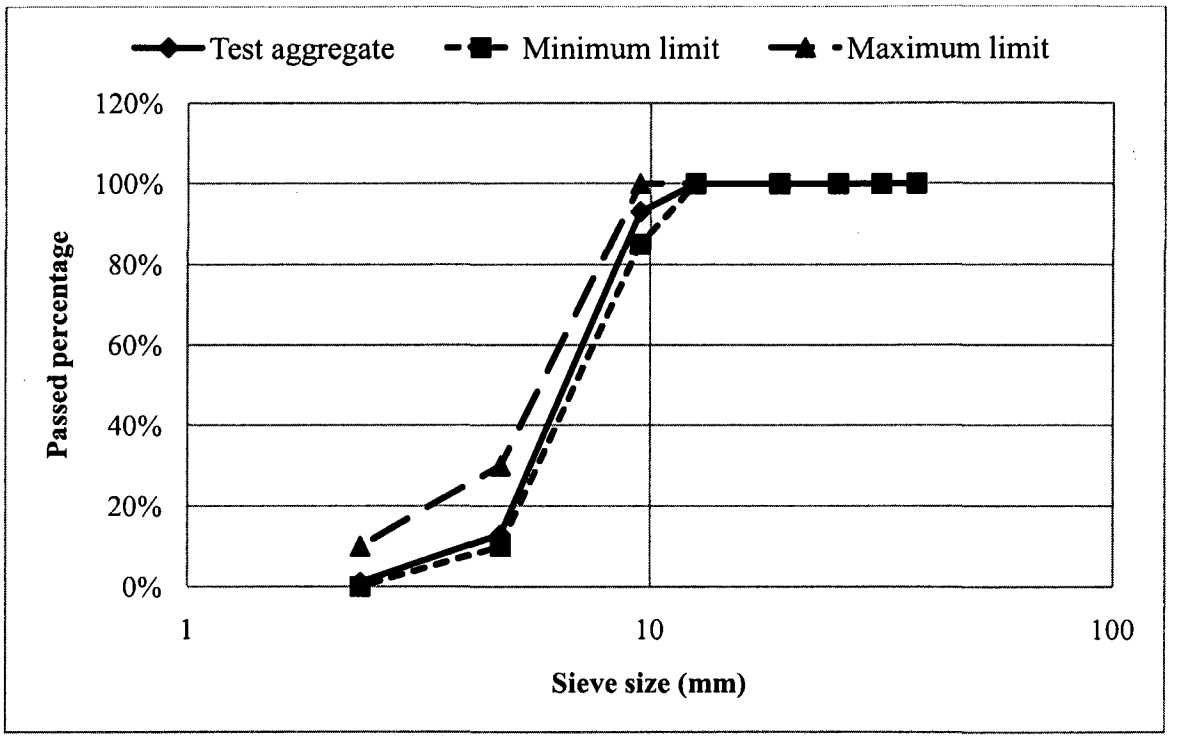

Figure 3.2 --- Sieve analysis of the coarse aggregate used in this study.

3.3.2 Specific gravity

Specific gravity of both the fine and coarse aggregates was determined according to ASTM C 128 (2007) and ASTM C 127 (2007), respectively and the results are presented in Table 3.3.

Table 3.3 --- Specific gravity of the fine and coarse aggregates used in this study.

Specific gravity	Fine aggregate	Coarse aggregate
Oven-dry	2.675	2.562
Saturated-surface-dry	2.692	2.529
Apparent	2.721	2.479

3.3.3 Absorption

Absorption of the fine and coarse aggregates was determined in accordance with ASTM C 128 (2007) and ASTM C 127 (2007), respectively. Two 200 litre barrels of the coarse aggregate were used in this research. The first one which was used in concrete mixture numbers one to four had water absorption of 0.95 %. The second barrel of the coarse aggregate with water absorption of 1.05 % was used in the last two concrete mixtures (numbers five and six). Only one barrel of the fine aggregate was used in this research for all six concrete mixtures with water absorption of 0.63 %.

3.3.4 Bulk density

According to ASTM C 29/C 29M (2007), bulk density of the fine and coarse aggregates were determined to be 1610 kg/m^3 and 1470 kg/m^3, respectively.

3.4 Mixture proportions

In this research, three different concrete types were made. Due to the concrete mixer capacity, two mixtures of each type were cast. Mixture numbers one and two were the

concrete made with fly ash at 25% replacement of cement. Mixture numbers three and four were the concrete made with slag at 35% replacement of cement. And finally mixture numbers five and six were the plain concrete. Mixture numbers one, three, and five were used to compare the ASTM C 672/C 672M (2003) with its proposed replacement method while mixture numbers two, four, and six were used to study the effects of a curing compound and a three-day curing method on the salt scaling resistance of concrete surfaces.

The mixture proportions used in this study were similar to the ones used by Bouzoubaa et al. (2008) to compare the results. As well, the cement replacements used are typical for the concrete industry. The mixture proportions for all six concrete mixtures used in this research are given in Table 3.4.

Table 3.4 --- Mixture proportions used in this research for 30 litres of concrete.

	Concrete made with fly ash		**Concrete made with slag**		**Plain concrete**	
	Mix #1	**Mix #2**	**Mix #3**	**Mix #4**	**Mix #5**	**Mix #6**
(w/c) ratio	0.40	0.40	0.40	0.40	0.40	0.40
Water, kg	4.8	4.8	4.8	4.8	4.8	4.8
Fine aggregate, kg	20.1	20.1	20.1	20.1	20.1	20.1
Coarse aggregate, kg	33.0	33.0	33.0	33.0	33.0	33.0
Cement, kg	9.0	9.0	7.8	7.8	12.0	12.0
Fly ash, kg	3.0	3.0	-	-	-	-
Slag, kg	-	-	4.2	4.2	-	-
Total Cementitious materials, kg	12.0	12.0	12.0	12.0	12.0	12.0
Air-entraining admixture, mL	11.0	11.0	15.0	14.0	10.0	12.0
Mid-range water-reducing admixture, mL	35.0	35.0	35.0	35.0	35.0	35.0
High-range water-reducing admixture, mL	40.0	55.0	60.0	50.0	85.0	65.0

3.5 Casting and curing of the test specimens

In this section, the casting procedure including concrete mixing, casting, and three different curing methods will be described in detail.

3.5.1 Concrete mixing and casting

Concrete mixing in this study was conducted in accordance with ASTM C 192/C 192M (2007). For the design a concrete mixture, the saturated surface dry condition of the aggregates is assumed. Since the aggregates may contain some moisture in their pores or on their surface, the moisture content of the aggregates should be determined to adjust the amount of water needed for the mixture proportion. In this study, the moisture content of both the fine and coarse aggregates was determined in accordance with ASTM C 566 (2004). One day before making each concrete mixture, both the fine and coarse aggregates were placed in an oven at a temperature of 110 ± 5 °C for sufficient time to reach a constant dry mass. Then the aggregates were cooled at room temperature for almost one hour and a half and the mass was measured. Then the moisture content of both the fine and coarse aggregates was determined and used to adjust the amount of water needed for the mixture. Due to the settling of the water to the bottom of the storage barrels, the moisture content of the aggregates varied throughout the depth of the barrels. The results for the moisture content of the aggregates for each concrete mixture are presented in Table 3.5.

Table 3.5 --- Moisture contents of the fine and coarse aggregates used in this study.

		Moisture content (%)	
		Fine aggregate	Coarse aggregate
Concrete made with fly ash	Mix #1	1.61	0.30
	Mix #2	2.35	0.69
Concrete made with slag	Mix #3	2.39	0.69
	Mix #4	0.76	0.29
Plain concrete	Mix #5	3.93	0.68
	Mix #6	5.74	0.69

After determining the moisture content of the fine and coarse aggregates, the amount of water needed for each concrete mixture was adjusted. The original and the adjusted amounts of water used in each concrete mixture are presented in Table 3.6.

Table 3.6 --- The original and the adjusted amounts of water used in each concrete mixture made in this study.

	Concrete made with fly ash		Concrete made with slag		Plain concrete	
	Mix #1	Mix #2	Mix #3	Mix #4	Mix #5	Mix #6
Original amount of water, kg	4.80	4.80	4.80	4.80	4.80	4.80
Adjusted amount of water, kg	4.82	4.54	4.53	4.99	4.26	3.89

Based on the required amount of materials for each concrete mixture as listed in section 3.4, cement, water, the fine and coarse aggregates in all concrete mixtures and fly ash or slag (in some concrete mixtures when required) were weighed and placed in separate buckets. Required amounts of chemical admixtures for each concrete mixture as listed in section 3.4 were placed in separate graduated cylinders.

The inside of the mixer was first dampened by a wet sponge to avoid any water absorption by the dry mixer. The entire mid-range water-reducing admixture and half of the high-range water-reducing admixture were added to the mixing water and were mixed thoroughly while the air-entraining admixture was added to the fine aggregate.

Coarse aggregate and some of the mixing water including water-reducing admixtures were added to the mixer. Then the mixer was started and the fine aggregate, cement, the rest of the water, and fly ash or slag (when required) were added to the mixer while it was running. After all ingredients were in the mixer, the concrete was mixed for three minutes, followed by a three-minute rest, followed by a two-minute final mixing period. The other half of the high-range water-reducing admixture was added to the concrete in the mixer during the three-minute rest period.

As required by ASTM C 672 and its proposed replacement method, the minimum surface area and depth for each scaling specimen should have been 0.045 m^2 and 75 mm, respectively. In this research plastic containers with a roughly rectangular shape having a surface area of 0.0554 m^2 and a depth of 90 mm were used for the scaling slabs. For each combination of variables to be tested two duplicate specimens were made.

Interior surfaces of the plastic slab molds were sprayed by a thin layer of WD-40 oil prior to fabrication of the specimens. The concrete was poured into the molds using a scoop in one layer as specified in ASTM C 672 and its proposed replacement method. Since it was required by the mentioned ASTM standards to rod one time for each 1400 mm^2 of the surface, forty times of rodding were conducted for each scaling specimen and a slight excess of material was left after the final rodding. The sides of the molds were

tapped to close any voids and the surface of the scaling specimens was levelled with several passes of a wood strike-off board (2" by 4" wooden board).

After the bleeding of the concrete stopped, the surface of the scaling specimens was finished with three sawing-motion passes of a wood strike-off board (2" by 4" wooden board) proceeded in a back-and-forth (horizontal sawing) motion with a ± 50 mm amplitude in both directions, along the length and along the width. This procedure was repeated until achieving a uniform surface free of holes and exempt of any aggregate while avoiding any excessive finishing (ASTM C 672/C 672M, 2003; the proposed replacement method for ASTM C 672).

At the end, the surface of the scaling specimens was brushed with a medium-stiff brush as the final finishing operation. In cases where used, the curing compound was sprayed on the surface of the scaling specimens after the brushing.

Three cylinders were prepared from each concrete mixture for the compression test according to ASTM C 39/C 39M (2005). In this research typical plastic cylinder molds with a nominal diameter of 100 mm and a height of 200 mm were used. A hole was prepared at the bottom of each cylinder to help release the samples at the time of the demoulding with compressed air. In addition, the interior of the cylinders were lightly sprayed with a release agent prior to casting.

Each cylinder was filled in two layers and each layer was rodded 25 times as specified in ASTM C 192/C 192M (2007). A plastic lid was used on the top of each cylinder to avoid the evaporation of the moisture from the surface of the cylinders.

3.5.2 Curing procedure

In this experimental research, three different curing procedures including the standard curing method, the three-day curing method and the use of a curing compound were employed which will be explained in more detail in the following sections.

3.5.2.1 The standard curing method

As specified in ASTM C 672 and its proposed replacement method, after the final finishing of the concrete specimens, they were immediately covered with a plastic sheet without any contact with the concrete surface. The slab specimens and cylinders were removed from the molds at an age of 20 to 24 hours after addition of water to the concrete mixture (beginning of casting) and were submerged in the moist storage as provided for in specification ASTM C 511 (2006). In this study, large plastic containers were used as storage tanks having a mixture of water and calcium hydroxide (lime). The concentration of calcium hydroxide was three grams in each litre of water; therefore for each container having 25 litres of water, 75 grams of calcium hydroxide were used.

The scaling specimens were removed from the moist storage at the age of 14 days and were stored in the air for 14 days at the laboratory temperature of 23 ± 2 °C while the cylinders were kept in the moist storage until the age of 28 days.

3.5.2.2 The three-day curing method

In this method, the same steps as the usual curing, specified in section 3.5.2.1, were followed except that the scaling specimens were removed from the moist storage at the

age of three days and were stored in the air for 25 days at the laboratory temperature of 23 ± 2 °C.

3.5.2.3 The curing compound

In this method, an evaporation reducer, Confilm®, was used as a curing compound to see the effect of a curing compound on the salt scaling resistance of concrete surfaces. The evaporation reducer was sprayed on the scaling specimen's top surface after the samples were prepared, the final finishing was done and the bleeding was stopped. The mass of the curing compound used for each scaling specimen was eight grams which were sprayed on the surface of each scaling specimen.

After spraying the evaporation reducer on the surfaces of the scaling specimens, they were kept in the laboratory. The slab specimens were removed from the molds at an age of 20 to 24 hours after addition of water to the concrete mixture (beginning of casting) and were stored in the air for 27 days at the laboratory temperature of 23 ± 2 °C.

3.6 Procedures of the tests

In this section, all the tests performed in this study will be described in detail. The tests conducted on fresh concrete included slump, unit weight, and air content and the tests performed on hardened concrete included compression test, and salt scaling test.

3.6.1 Fresh concrete tests

After mixing each concrete mixture, the slump test was carried out in accordance with ASTM C 143/C 143M (2008) to make sure that the slump was within the required range

(75 ± 15 mm). If the slump was lower than the minimum required limit, more high-range water-reducing admixture (Glenium® 7500) was added to the mixture. The concrete was mixed for another one minute and the slump test was performed again. This procedure was repeated until the slump was within the required range.

After finishing the slump test, unit weight of each concrete mixture was determined in accordance with ASTM C 138/C 138M (2008).

After obtaining an adequate slump, the air content of each concrete mixture was measured according to ASTM C 231 (2008) to make sure that the air content was within the required range (6 ± 1 %). In cases where the air content of the concrete mixture was not in the required range (lower than 5% or higher than 7%), that mixture was abandoned and another mixture with more or less air-entraining admixture was made to have the air content within the required range.

3.6.2 Compression test

At the age of 28 days, a compression test was conducted on all three cylinders of each concrete mixture in accordance with ASTM C 39/C 39M (2005). Cylinders were removed from the moist storage tanks at the age of 28 days and were transported in a mixture of water and calcium hydroxide to the structural lab to perform the compression test. Before starting the test, the specimens were cleaned by a moist towel, and their height and diameters (both the rough and smooth surfaces of each cylinder) were measured. Then the specimens were placed hardened face up between two load bearing caps (Figure 3.3) to distribute the load equally all over the surface of specimens, because

the cast face of the specimens usually does not have a completely smooth surface. Inside of the caps, there was a hard thick plastic which could be replaced after several uses.

The cylinders with the caps were placed on the compression machine and compressive axial load was applied until the failure occurred (Figure 3.3). ASTM C 39/ C39M (2005) requires a uniform rate of loading; however, in this research the rate of loading was visually controlled in a continuous rate without any shocks applied to the specimens since the machine used in this study did not have the capability for rate control. Failure load was recorded for each cylinder and photographs of the failed specimens were taken.

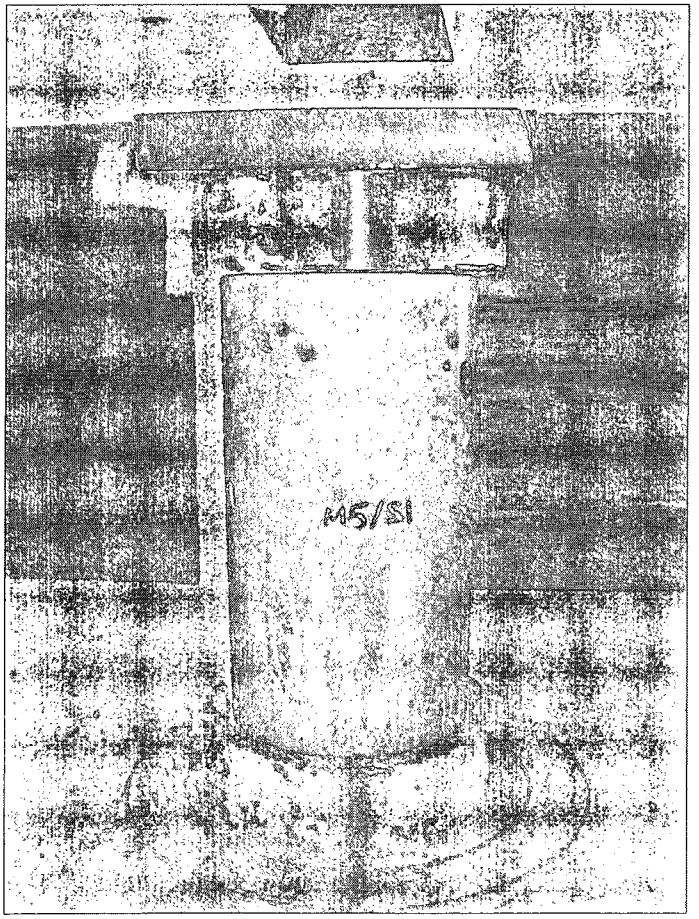

Figure 3.3 --- Set-up for compression test.

3.6.3 Salt scaling test

The salt scaling resistance of concrete surfaces was evaluated according to two different procedures, ASTM C 672/C 672M (2003) and its proposed replacement method which will be described in detail in the following sections.

For testing the scaling resistance, calcium chloride solution is ponded on the top surface of the cast and cured specimens. To maintain a pond on top of the specimen throughout the freezing-and-thawing cycles, a flexible plastic material (designed for use as flooring baseboard) with thickness of 3 mm and height of 65 mm was adhered to the scaling specimen with silicon based glue along the perimeter of the top surface of the specimens (Figures 3.4 and 3.5). Approximately 40 mm of the plastic baseboard projected beyond the top of the specimen's surface to maintain enough height for the saline solution (6 mm depth) and to prevent splashing on transport of specimens. To avoid any possible leakage of the saline solution from the corners where the plastic baseboard was adhered to the specimen, another type of silicon based sealant was used at the corners and at the bottom of the baseboard.

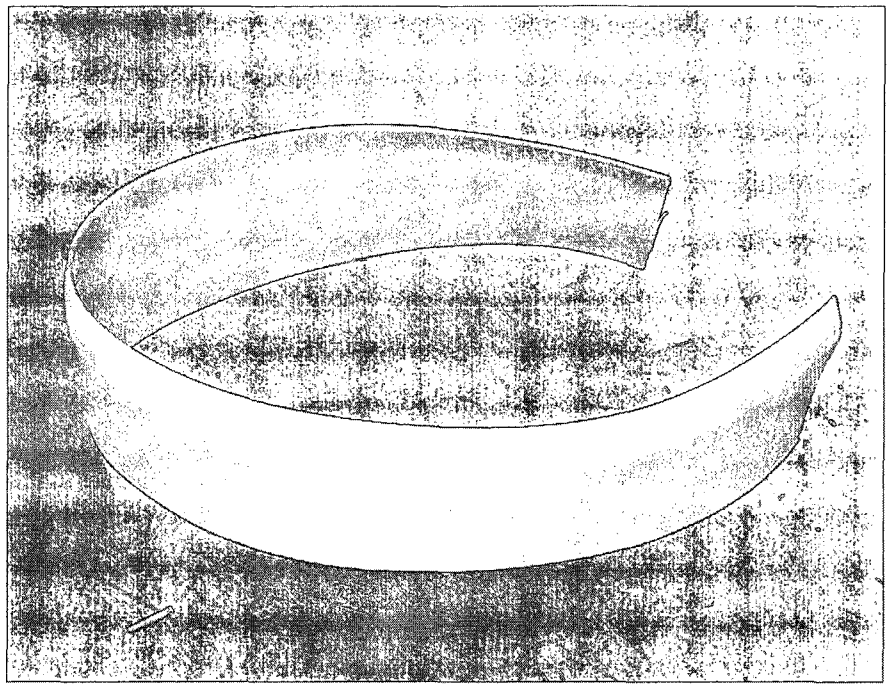

Figure 3.4 --- Plastic baseboard used in this study.

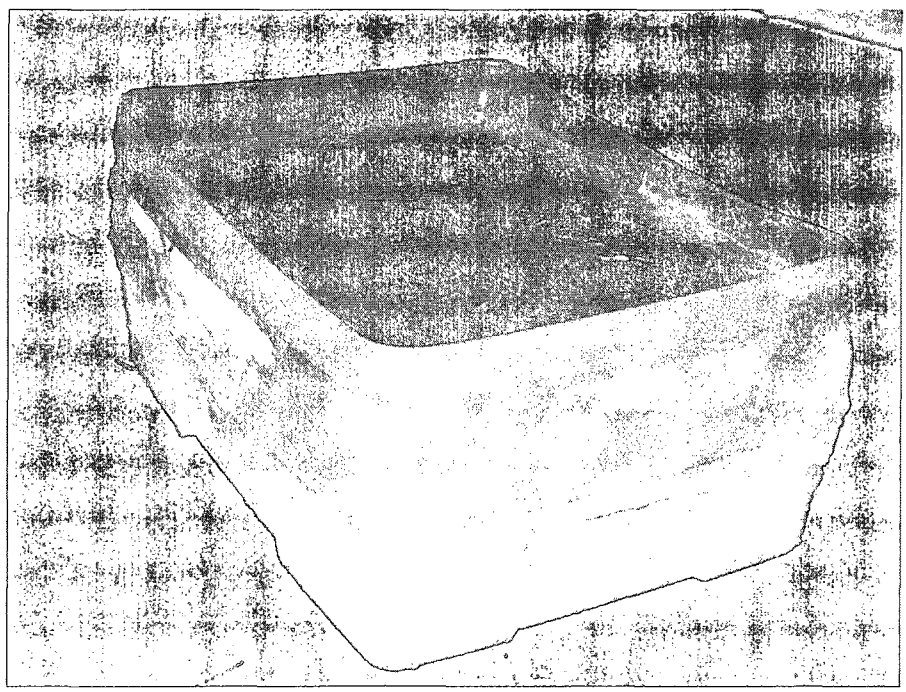

Figure 3.5 --- Scaling specimen with the plastic baseboard adhered to the top surface used in this research in the freezing-and-thawing cycles.

3.6.3.1 ASTM C 672

As specified in ASTM C 672, after completion of the moist and air curing, the flat surface of the scaling specimens was covered with approximately 6 mm of calcium chloride solution as described in section 3.2.6 and the freezing-and-thawing cycles were started. First, the scaling specimens were placed in a freezer for 16 to 18 hours. As specified in ASTM C 672 and its proposed replacement method, the freezing equipment should be capable of lowering the temperature of the scaling specimens to -18 ± 3 °C within 16 to 18 hours and maintaining this temperature with a full load of specimens. To determine the temperature at the interface of the concrete surface and the saline solution and also inside the freezer, a temperature data-logger was placed in different positions both at the interface of specimens' surfaces and saline solution and also on different shelves inside the freezer. The typical results of measured temperature are presented in Figures 3.6 and 3.7.

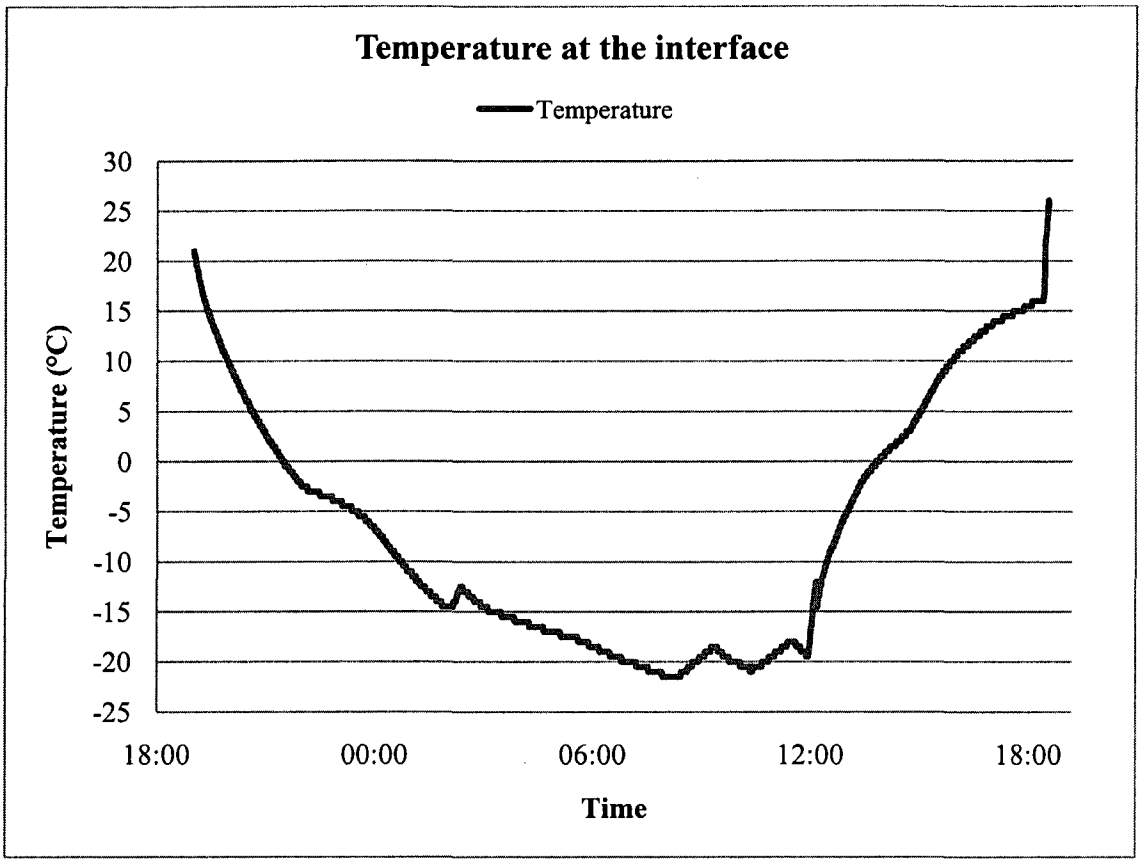

Figure 3.6 --- Temperature at the interface of the concrete surface and the saline solution for one freezing-and-thawing cycle.

It can be seen in Figure 3.6 that the temperature at the interface reached the required temperature (-18 ± 3 °C) after approximately eight hours (the freezing cycle started at 19:00 hours and the temperature reached -15 °C at 03:07 hours on the next day). The sample was removed from the freezer at the end of 17-hour freezing period (at 11:56 hours in Figure 3.6) after which the temperature was increased. The scaling specimen was placed in the laboratory at 23 ± 2 °C for another 7 hours (thawing period) to complete a full freezing-and-thawing cycle (24 hours).

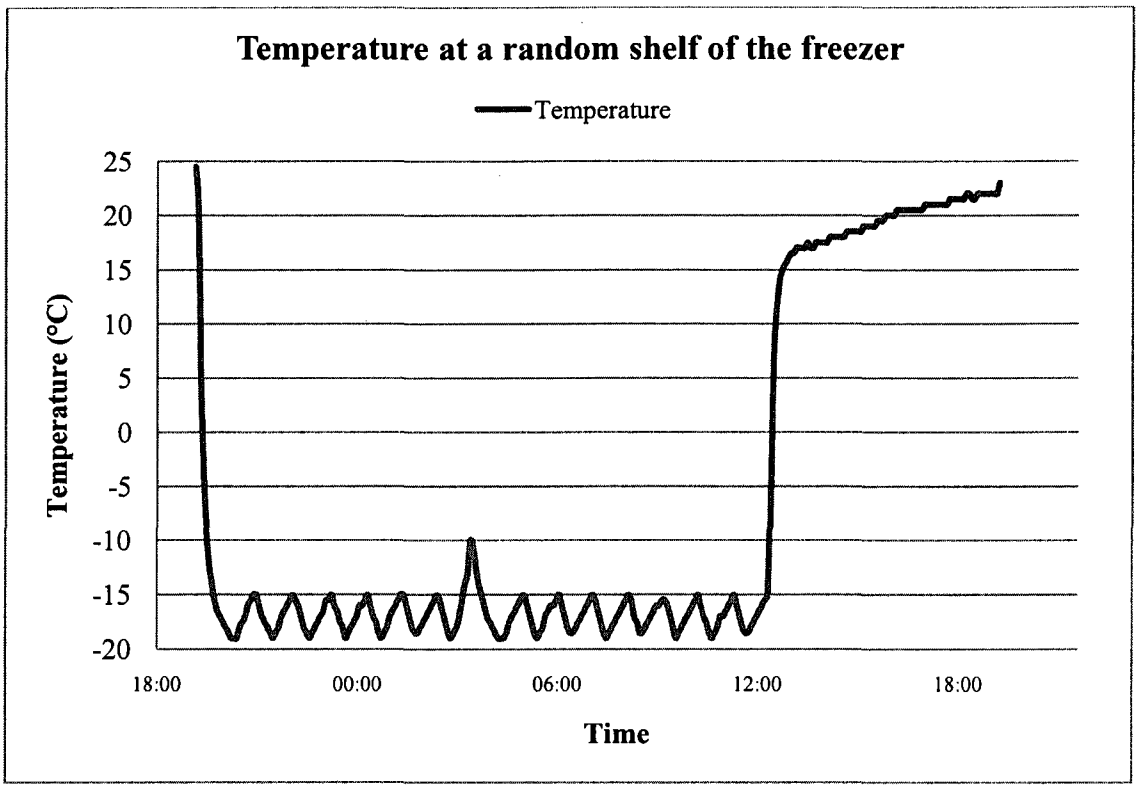

Figure 3.7 --- Temperature at a random shelf of the freezer for one freezing-and-thawing cycle.

It can be seen in Figure 3.7 that the temperature at a random shelf of the freezer reached the required temperature (-18 ± 3 °C) after approximately less than an hour (the freezing cycle started at 19:10 hours and the temperature reached -15.5 °C at 19:43 hours). It should be mentioned that for the temperature measurement at a random shelf, the first spike to -10° C and the varying temperature between -15 °C and -20° C was due to the temperature cycling of the freezer and was unavoidable. The specimen was removed from the freezer after 17 hours (at 12:21 hours in Figure 3.7) after which the temperature was increased. The scaling specimen was placed in the laboratory at 23 ± 2 °C for another 7 hours (thawing period) to complete a full freezing-and-thawing cycle (24 hours).

At the end of the freezing period, the scaling specimens were removed from the freezer and then were placed in the laboratory air at 23 ± 2 °C for 6 to 8 hours. Between each cycle water was added to the solution on the surface of the specimens to maintain the proper depth of the solution due to the water evaporation during the thawing period (6 to 8 hours in the room temperature). In case of any leakage of the saline solution, instead of adding water, additional saline solution was prepared and added to the solution on the surface of the scaling specimens. This procedure was repeated daily. In the one case where the cycling schedule could not be maintained, scaling specimens were kept frozen until thawing could occur. This event happened only once for one day in this study (all four scaling specimens of concrete mixture number four and only two out of four scaling specimens of mixture number five). The lengthened cycle had 41.5 hours of freezing (instead the usual 16 to 18 hours), but was still counted as one cycle as per the directions in the standard.

At the end of each five cycles, the surface of the scaling specimens was flushed off thoroughly with the same saline solution specified in section 3.2.6 to remove all flaked off particles. The scaling residue was collected on a pre-weighed filter paper. Then the filter paper with the scaling residue was placed in an oven having a temperature of 110 °C for 24 hours. At the end of the oven drying period, the filter paper with the scaling residue was removed from the oven and was weighed again. The difference in the mass of the empty filter paper and the mass of the filter paper with the scaling residue was measured to be the mass of the scaling residue from the surface after every five cycles. The surface area exposed to the saline solution was measured to the nearest one square

centimetre as specified in section 3.5.1. Therefore the mass loss in kg/m^2 was determined by dividing the mass of scaling residue by the exposed surface area.

Photographs of the scaled surface of the specimens were taken and after a visual examination of the surface, a visual rating was reported for each scaling specimen according to Table 3.7. After these processes, the surface of the scaling specimens was covered by the same saline solution specified in section 3.2.6 and the freezing-and-thawing cycles were resumed.

Table 3.7 --- Visual rating in accordance with ASTM C 672/C 672M (2003).

Rating	Condition of surface
0	No scaling
1	Very slight scaling (3 mm depth, max, no coarse aggregate visible)
2	Slight to moderate scaling
3	Moderate scaling (Some coarse aggregate visible)
4	Moderate to severe scaling
5	Severe scaling (Coarse aggregate visible over entire surface)

As specified in ASTM C 672, generally 50 freezing-and-thawing cycles are sufficient to evaluate a surface or surface treatment. Therefore in this research the freezing-and-thawing cycles were repeated until 50 cycles had been completed. The mass loss and visual rating of the concrete surfaces were determined after every five cycles.

It should be mentioned that ASTM C 672 requires only visual rating and does not include measurement of the mass loss. But in this research to have a comparison between

ASTM C 672 and its proposed replacement method, the mass loss was also determined in addition to the visual rating in both methods after every five cycles. Also both ASTM C 672 and its proposed replacement method require the visual rating and flushing off the surface after 5, 10, 15, 25, and every 25 cycles thereafter but in this study to have more results this procedure was done after every 5 cycles until 50 cycles.

3.6.3.2 The proposed replacement method for ASTM C 672 (WK 9367)

The same procedure as ASTM C 672 was conducted in its proposed replacement method. The only difference was that in the proposed replacement method, after completion of the moist and dry curing, instead of starting the freezing-and-thawing cycles as in ASTM C 672, the surface of the scaling specimens was pre-saturated with a layer of 5 mm ± 2 mm of the same saline solution as specified in section 3.2.6. The period of the pre-saturation was seven days and the surface of the scaling specimens was covered with a plastic sheet not touching the surface to prevent any evaporation of the solution. At the end of the seven days pre-saturation period, the freezing-and-thawing cycles were started and resumed until completion of 50 cycles. All the details about flushing off the surface and determining the mass loss in the proposed replacement method were as the same as ASTM C 672 except the visual rating which was according to a different scale as given in Table 3.8.

Table 3.8 --- Visual rating according to the proposed replacement method for ASTM C 672.

Category	Characteristics of the scaling surface
0	No significant scaling observed
1-A	Slight scaling of the surface mortar with no popouts *
1-B	Important scaling of the surface mortar with no popouts
2-A	No significant scaling of the surface mortar, but presence of few popouts
2-B	No significant scaling of the surface mortar, but presence of several popouts
3	Combination of scaling of the surface mortar mainly with cracked coarse aggregate
4	Combination of cracked coarse aggregate mainly with scaling of the surface mortar

*The causes for the formation of popouts are: a) The rupture of the aggregate b) The sudden detachment of the mortar over the aggregate

Chapter 4

Experimental results

4.1 General

In the previous chapter, an experimental program to study the salt scaling resistance of concrete surfaces exposed to the freezing-and-thawing cycles in the presence of de-icing chemicals was described. Three concrete types (two mixtures of each) including plain concrete, and concretes with 25% replacement of cement with fly ash and 35% with slag were made. Eighteen cylinders (three specimens from each concrete mixture) were prepared and tested to determine the compressive strength of each concrete mixture at the age of 28 days. Totally 24 slab specimens (four specimens from each concrete mixture) for all the different variables studied in this research were prepared and tested to study the salt scaling resistance of concrete surfaces.

In the following sections, the results of the experimental investigation are reported and discussed. All the tests were conducted according to ASTM standards which are explained in detail in the previous chapter. First the results of fresh concrete tests including slump, unit weight, and air content will be presented followed by the compressive strength results at the age of 28 days for all concrete mixtures. Thereafter, the results of salt scaling test will be shown which include the comparison of ASTM C 672/C 672M (2003) with its proposed replacement method, the comparison of the salt scaling resistance of the plain concrete and the concretes with 25% replacement of

cement with fly ash and 35% with slag, and study the effects of a specific curing compound and a three-day curing method on salt scaling resistance.

4.2 Experimental results of fresh concrete

In this section, experimental results of tests conducted on fresh concrete including slump, unit weight, and air content are described and discussed (Table 4.1).

Table 4.1 --- Test results of fresh concrete for all concrete mixtures.

		Slump, mm	Unit weight, kg/m^3	Air content (%)
Concrete made with fly ash at 25% replacement of cement	**Mix #1**	75	2,237	6.5
	Mix #2	65	2,268	5.5
Concrete made with slag at 35% replacement of cement	**Mix #3**	80	2,257	7.0
	Mix #4	60	2,238	7.0
Plain concrete	**Mix #5**	75	2,328	5.0
	Mix #6	85	2,296	5.5

As mentioned in section 3.6.1, the slump test was conducted according to ASTM C 143/C 143M (2008) after making each of six concrete mixtures in this research. The experimental results of slump are listed in Table 4.1.

The slump test results of all six concrete mixtures varied from 60 to 85 mm which are all within the required range (75 ± 15 mm) by both ASTM C 672 and its proposed replacement method. Although the amount of the mid-range water-reducing admixture was constant for all six concrete mixtures (35 mL for 30 litres of concrete or 292 mL/100kg of total cementing material), the amount of the high-range water-reducing admixture varied in each concrete mixture to keep the slump within the required range.

As mentioned in section 3.6.1, unit weight of each concrete mixture made in this research was determined according to ASTM C 138/C 138M (2008) to the nearest 1.0 kg/m^3. The experimental results of unit weight are also given in Table 4.1.

The unit weight of concretes made in this research varied from 2,237 to 2,328 kg/m^3 which are all found to be within the expected range of concrete's unit weight. The range of unit weight of conventional concrete, normally used in pavements, buildings, and other structures is 2,200 to 2,400 kg/m^3 and in the design of reinforced concrete structures, the unit weight of the combination of conventional concrete and reinforcing bars is assumed to be 2,400 kg/m^3 (Kosmatka et al., 2002).

As mentioned in section 3.6.1, after obtaining an adequate slump, the air content of each concrete mixture was measured in accordance to ASTM C 231 (2008). As it can be seen in Table 4.1, the air content of all six concrete mixtures made in this study varied from 5.0 to 7.0 % which are all within the required range (6 ± 1 %) by both ASTM C 672 and its proposed replacement method.

4.3 Experimental results of compressive strength

As mentioned in section 3.6.2, the compression test on all three cylinders of each concrete mixture was conducted according to ASTM C 39/C 39M (2005) at the age of 28 days. The maximum load at failure derived from the compression testing machine was converted from pound force (lbf) to Newton (N) by multiplying the loads by 4.448222. The cylindrical specimens in this study had a nominal diameter of 100 mm and a height of 200 mm. Therefore the cross-section area of each cylindrical specimen was calculated from equation 4.1.

$$A = \pi \times (D/2)^2 = \pi \times (100/2)^2 = 7854 \text{ mm}^2 \quad (4.1)$$

The compressive strength (σ) of each cylindrical specimen in MPa was calculated by division of the load (P) in Newton by the area (A) in mm^2 as specified in equation 4.2.

$$\sigma \text{ (MPa)} = P \text{ (N)} / A \text{ (mm}^2\text{)} \quad (4.2)$$

The compressive strength of all six concrete mixtures was determined and rounded to the nearest 0.1 MPa according to ASTM C 39/C 39M (2005). The experimental results of the compressive strength, the coefficient of variation, and the standard deviation are given in Table 4.2. The results of each mixture represent the average of the three specimens cast. Since each mixture was cast twice due to mixer capacity constraints, the average of the two mixtures is also given.

Table 4.2 --- Compressive strength test results of all concrete mixtures at 28 days.

		Maximum load, KN	**Compressive strength, MPa**		**COV (%)**	**Standard deviation, MPa**
			Each mixture	Average		
Concrete made with fly ash at 25% replacement of cement	**Mix #1**	374	47.7	46.1	3.1	1.46
	Mix #2	350	44.6		2.3	1.04
Concrete made with slag at 35% replacement of cement	**Mix #3**	434	55.2	51.7	0.7	0.41
	Mix #4	378	48.1		0.3	0.15
Plain concrete	**Mix #5**	461	58.7	54.6	0.9	0.55
	Mix #6	396	50.5		1.8	0.92

It can be concluded from Table 4.2 that among all three concrete types, the plain concrete showed the highest average compressive strength followed by the concrete made with slag, followed by the concrete made with fly ash. These results were expected since concretes made with fly ash usually obtain strength at the later ages and the compressive strength of the concretes made with slag is almost as the same as the one of the plain concrete. It can be seen in Table 4.2 that there are slight differences in the compressive strength of each two mixtures in every concrete type. Since only three cylinders from each concrete mixture were prepared and tested to determine the compressive strength, there could be a slight difference in the compressive strength. When the number of samples in each concrete mixture increases, the average compressive strength of each mixture in every concrete type will vary less than it was observed in this study.

Acceptable value of the coefficient of variation for compressive strength of the concrete according to ASTM C 39/C 39M (2005) is 3.2 % for 100 by 200 mm cylinders. The coefficient of variation of all concrete mixtures presented in Table 4.2 found to be less than the maximum acceptable value (3.2%).

4.4 Experimental results of salt scaling test

In this section, the results of salt scaling test for all three concrete types will be presented and compared. First, the visual rating of the scaling specimens will be presented in accordance to ASTM C 672/C 672M (2003) and its proposed replacement method after each five cycles. Then the relationship between mass loss and freezing-and-thawing cycles will be presented to compare all of the variables studied in this research.

As mentioned previously, three types of concrete were cast in two batches. The first mixture of each pair tested the current ASTM C 672 standard and its proposed replacement method. The second mixture tested the curing variables (curing compound and three-day curing) subject to the proposed standard method.

4.4.1 Visual rating of the scaling specimens

In this section, the visual rating of the scaling specimens after each five cycles will be presented in accordance with ASTM C 672 and its proposed replacement method as specified in Tables 3.7 and 3.8 in sections 3.6.3.1 and 3.6.3.2, respectively. It should be mentioned that this visual rating is found to be very subjective. The mass loss can show a better view of the salt scaling resistance, as will be shown later.

4.4.1.1 According to ASTM C 672/C 672M (2003)

A visual rating of the scaling specimens after every five cycles was conducted according to ASTM 672 as specified in Table 3.7 in section 3.6.3.1 and the results are given in Table 4.3. As it was mentioned before this visual rating is very subjective.

Table 4.3 --- Visual rating of the scaling specimens according to ASTM C 672.

		Visual rating after cycle #									
		5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
Concrete made with fly ash	1st sample	3	3	3	3	3	3	3	3	3	5
	2nd sample	3	3	3	3	3	3	3	3	3	3
Concrete made with slag	1st sample	3	3	3	3	3	3	3	3	3	3
	2nd sample	3	3	3	3	3	3	3	3	3	3
Plain concrete	1st sample	3	3	3	3	3	4	4	4	4	5
	2nd sample	3	3	3	3	3	4	4	4	4	5

Two replicate specimens were tested for each of the three concrete types investigated. It can be seen in Table 4.3 that at the end of the first five cycles, moderate scaling (level 3) was observed in all concrete types. The visual rating remained constant (moderate scaling, level 3) in the fly ash specimens throughout the test period until the end of the 50th cycle with the exception of one specimen which showed severe scaling (level 5) only at the end of the 50th freezing-and-thawing cycle. The visual rating remained constant (moderate scaling, level 3) as well in the slag specimens throughout the test period until the end of the 50th cycle. For the plain concrete samples, the visual rating increased to 4 (moderate to severe scaling) after 30 cycles and to 5 (severe scaling) at 50 cycles. Photographs of the samples are shown later.

4.4.1.2 According to the proposed replacement method for ASTM C 672

A visual rating of the scaling specimens after every five cycles was conducted according to the proposed replacement method for ASTM 672 as specified in Table 3.8 in

section 3.6.3.2 and is given in Tables 4.4, 4.5, and 4.6. For each concrete type, two replicate specimens were tested for each of the three curing methods investigated. The visual rating in accordance with the replacement method for ASTM C 672 was found to have a limited use, since only three categories (0, 1-A, and 1-B) were usable in this study; no cracking was observed (cracking are linked to unsuitable aggregate rather than concrete properties). In some scaling specimens, few or several popouts were observed. Since there is no category for the combination of scaling and popouts in the proposed replacement method for ASTM 672, the numbers in the parentheses right after the categories (1-A or 1-B) in Tables 4.4, 4.5, and 4.6 refer to the presence of popouts (1 for few and 2 for several) combined with the surface scaling.

Table 4.4 --- Visual rating of the scaling specimens of the concrete made with fly ash according to the proposed replacement method for ASTM C 672.

		Visual rating after cycle #									
		5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
Standard curing	1st sample	1-A	1-A	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	1-A	1-A	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
Curing compound	1st sample	1-A	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	1-A	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
Three-day curing method	1st sample	1-A (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	1-A (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)

(1) : Few popouts (2) : Several popouts

It can be seen in Table 4.4 that at the end of the first five cycles slight scaling (category 1-A) was observed in all curing scenarios for the fly ash mixture. The visual rating was increased to important scaling (category 1-B) at the end of the 15th cycle in the standard curing method and at the end of the 10th cycle in both the method using the curing compound and the three-day curing method. Then the visual rating remained constant (important scaling, category 1-B) throughout the test period until the end of the 50th cycle. Photographs of the samples are shown later.

Table 4.5 --- Visual rating of the scaling specimens of the concrete made with slag according to the proposed replacement method for ASTM C 672.

		Visual rating after cycle #									
		5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
Standard curing	1st sample	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A
	2nd sample	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A	1-A
Curing compound	1st sample	0	0	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	0	0	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
Three-day curing method	1st sample	0	0	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	0	0	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)

(1) : Few popouts (2) : Several popouts

It can be seen in Table 4.5, for the slag mixtures, that at the end of the first five cycles slight scaling (category 1-A) was observed in the standard curing method which remained

constant (slight scaling, category 1-A) throughout the test period until the end of the 50th cycle. On the other hand, at the end of the first five cycles no significant scaling (category 0) was observed in both the method using the curing compound and the three-day curing method which was increased to important scaling (category 1-B) at the end of the 15th cycle and then was remained constant (important scaling, category 1-B) throughout the test period until the end of the 50th cycle in both methods. Photographs of the samples are shown later.

Table 4.6 --- Visual rating of the scaling specimens of the plain concrete according to the proposed replacement method for ASTM C 672.

		Visual rating after cycle #									
		5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
Standard curing	1st sample	1-A	1-A	1-A	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
	2nd sample	1-A	1-A	1-A	1-B (1)	1-B (1)	1-B (2)	1-B (2)	1-B (2)	1-B (2)	1-B (2)
Curing compound	1st sample	1-A	1-A	1-A	1-A	1-A	1-B	1-B (1)	1-B (1)	1-B (1)	1-B (1)
	2nd sample	1-A	1-A	1-A	1-A	1-A	1-B	1-B (1)	1-B (1)	1-B (1)	1-B (1)
Three-day curing method	1st sample	1-A	1-B	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)
	2nd sample	1-A	1-B	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)	1-B (1)

(1) : Few popouts (2) : Several popouts

It can be seen in Table 4.6 that at the end of the first five cycles slight scaling (category 1-A) was observed in all three different curing methods for the plain concrete.

The visual rating was increased to important scaling (category 1-B) at the end of the 20^{th} cycle in the standard curing method, at the end of the 30^{th} cycle in the method using the curing compound and at the end of the 10^{th} cycle in the three-day curing method. Then the visual rating was remained constant (important scaling, category 1-B) throughout the test period until the end of the 50^{th} cycle for all three different curing methods. Photographs of the samples are shown later.

4.4.2 Photographs of the scaling specimens after 50 freezing-and-thawing cycles

The photographs of the scaling specimens were taken at the end of 50 freezing-and-thawing cycles which are presented in Appendix C.

4.4.3 Comparison of the mass loss results

In this section, the mass loss results of the scaling tests for each variable studied in this research will be presented and compared. First ASTM C 672/C 672M (2003) will be compared to its proposed replacement method. Then the effects of curing (standard, curing compound and three-day curing methods) on salt scaling resistance will be studied. Finally the salt scaling resistance of the plain concrete and the concretes made with 25% replacement of cement with fly ash and 35% with slag will be compared in accordance to all variables studied in this research. It should be mentioned that the mass loss results are an average of the two replicate specimens regarding each variable and all the detailed results for each scaling specimen are included in the Appendix B.

4.4.3.1 Comparison of ASTM C 672 with its proposed replacement method

In this section, the results of mass loss for different concrete mixtures including the plain concrete, and the concretes made with 25% replacement of cement with fly ash and 35% with slag are presented according to ASTM C 672 and its proposed replacement method. The relationship between mass loss and the number of freezing-and-thawing cycles for all three types of concrete mixtures made in this research is presented in Figures 4.1, 4.2, and 4.3 in accordance with both ASTM C 672 and its proposed replacement method.

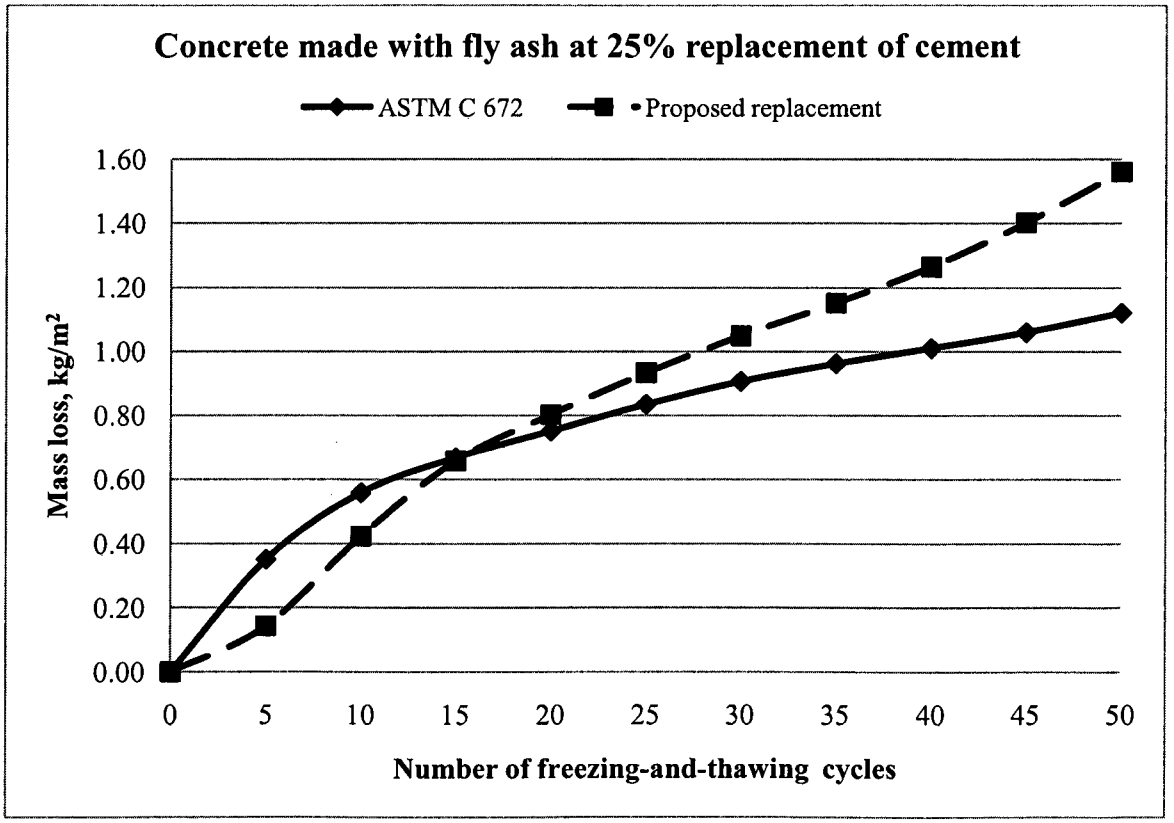

Figure 4.1 --- Mass loss versus number of freezing-and-thawing cycles in the concrete made with fly ash.

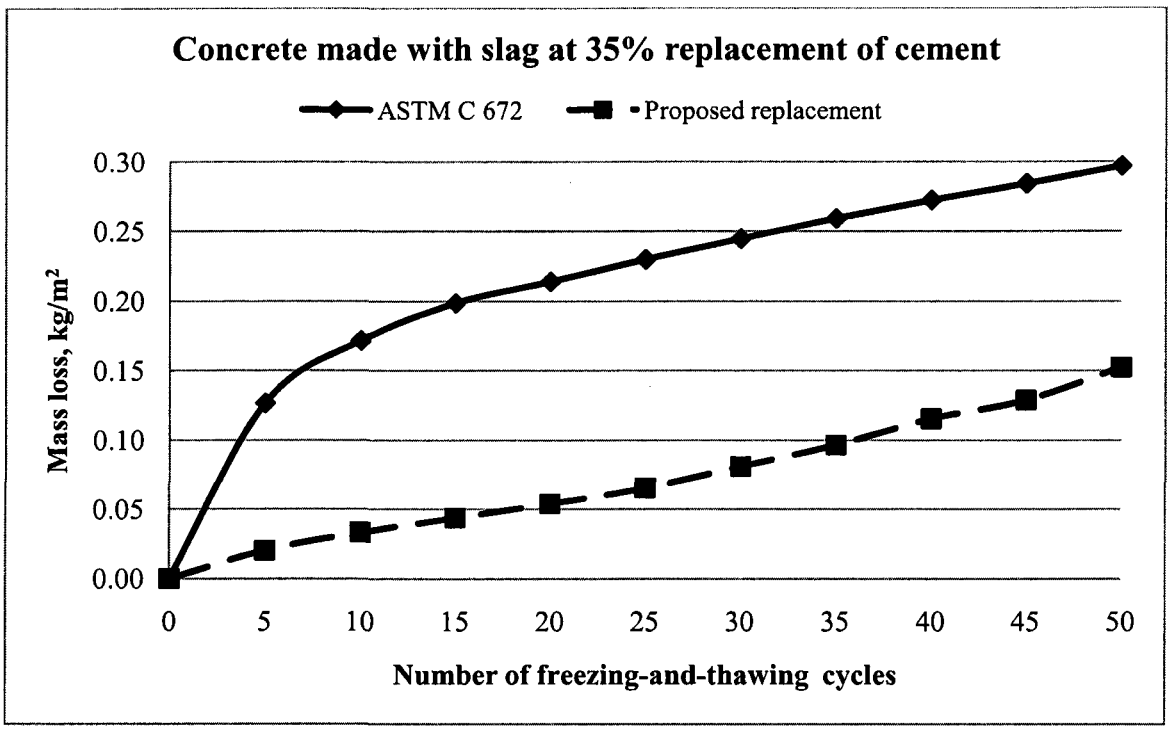

Figure 4.2 --- Mass loss versus number of freezing-and-thawing cycles in the concrete made with slag.

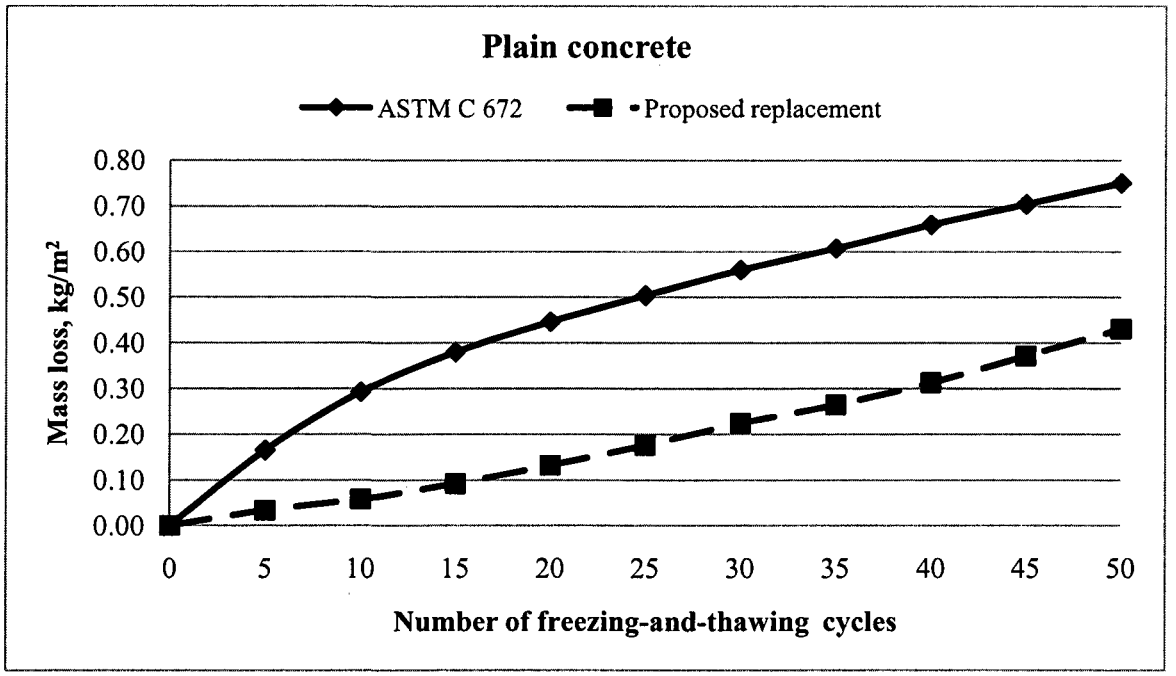

Figure 4.3 --- Mass loss versus number of freezing-and-thawing cycles in the plain concrete.

As it can be observed in Figures 4.2 and 4.3, ASTM C 672 showed greater mass loss than its proposed replacement method in both the plain concrete and the concrete made with slag (74% more in the plain concrete and 95% more in the concrete made with slag at the end of 50 freezing-and-thawing cycles). In the concrete made with fly ash (Figure 4.1), the mass loss of the replacement method was less until the 15^{th} cycle. However, after cycle #15, the proposed replacement method showed greater mass loss (39% more at the end of 50 freezing-and-thawing cycles) than ASTM C 672. Although in the concrete made with fly ash (Figure 4.1) the proposed replacement method started out with less mass loss than ASTM C 672, the rate of increase was high throughout which led to a greater mass loss at the end of 50 freezing-and-thawing cycles.

Greater mass loss results in lower salt scaling resistance. Therefore, it can be concluded that the salt scaling resistance of the concrete made with slag as well as the plain concrete according to the proposed replacement method was higher than ASTM C 672. But in the concrete made with fly ash, based on the final mass loss at the end of 50 freezing-and-thawing cycles, it can be concluded that ASTM C 672 showed higher salt scaling resistance than its proposed replacement method.

4.4.3.2 The effects of curing on salt scaling resistance

In this section, the results of salt scaling test for each of three concrete types made in this research are presented separately to study the effects of curing on salt scaling resistance. The relationship between mass loss and freezing-and-thawing cycles for each concrete type in accordance to all variables studied in this research is presented in Figures 4.4, 4.5, and 4.6. These variables included standard 14-day moist curing, curing

compound, and three-day moist curing method. All tests were carried out using the proposed replacement method. The differences in these three methods are regarding the curing. In the standard 14-day moist curing, as specified in section 3.5.2.1, the specimens were cured in the moist storage until the age of 14 days while in the three-day moist curing method, as specified in section 3.5.2.2 the specimens were cured in the moist storage until the age of 3 days. At the end of the moist curing in both mentioned method, the scaling specimens were cured in the air until the age of 28 days. But in the method using the curing compound, the scaling specimens were cured only in air until the age of 28 days. At the end of the curing period (at the age of 28 days), the surface of the scaling specimens in all three methods were pre-saturated with the saline solution for seven days before starting the salt scaling test.

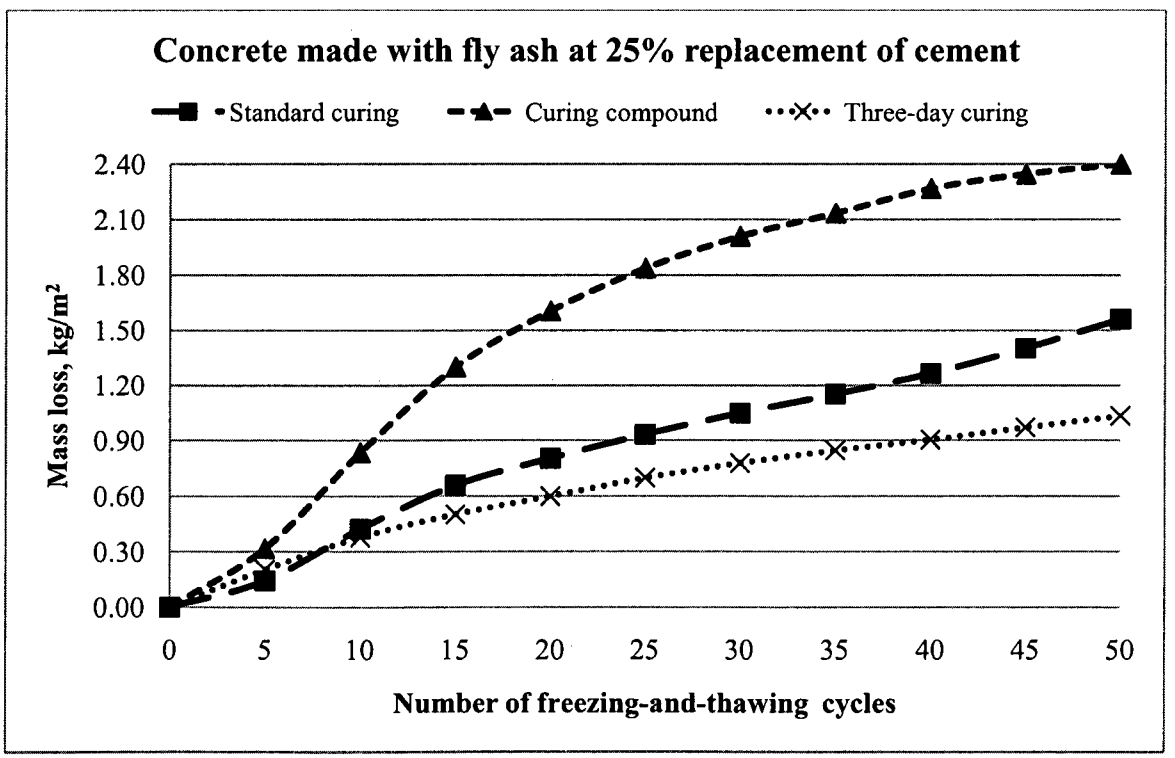

Figure 4.4 --- Mass loss comparison of different curing methods in the concrete made with fly ash.

As it can be observed in Figure 4.4, in the concrete made with fly ash in this study, using the curing compound, as specified in section 3.2.5, resulted in greater mass loss than the standard 14-day moist curing method (54% more at the end of 50 freezing-and-thawing cycles).

On the other hand, the three-day moist curing method resulted in less mass loss than the standard 14-day moist curing method (34% less at the end of 50 freezing-and-thawing cycles). Only prior to the 10^{th} cycle, the mass loss of the three-day moist curing method was slightly more than standard 14-day moist curing method. It also can be observed that the three-day moist curing method showed less mass loss than the method using the curing compound (57% less at the end of 50 freezing-and-thawing cycles).

Therefore it can be concluded that in the concrete made with fly ash, using this particular curing compound, as specified in section 3.2.5 with the mentioned dosage, resulted in lower salt scaling resistance than the standard 14-day moist curing method, while the three-day moist curing method showed higher salt scaling resistance than both the standard 14-day moist curing method and the method using the curing compound.

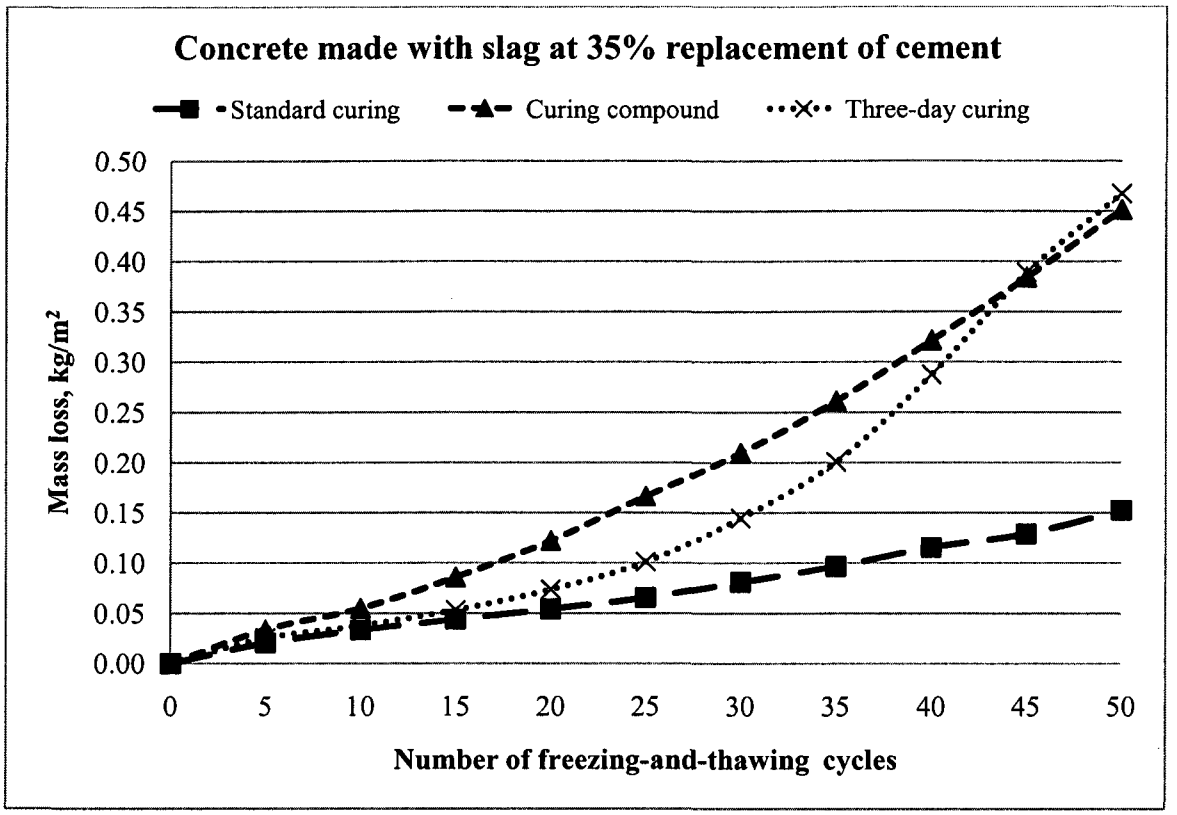

Figure 4.5 --- Mass loss comparison of different curing methods in the concrete made with slag.

As it is shown in Figure 4.5, in the concrete made with slag in this study, using the curing compound, as specified in section 3.2.5, as well as the three-day moist curing method resulted in greater mass loss (196% and 207% more, respectively at the end of 50 freezing-and-thawing cycles) than the standard 14-day moist curing method. The method using the curing compound also showed more mass loss than the three-day moist curing method until the 45th cycle but the result was slightly opposite after cycle #45 where at end of 50 freezing-and-thawing cycles, the three-day moist curing method showed 4% greater mass loss.

Therefore it can be concluded that in the concrete made with slag, using this particular curing compound, as specified in section 3.2.5 with the mentioned dosage, as well as the

three-day moist curing method resulted in lower salt scaling resistance than the standard 14-day moist curing method. Also the three-day moist curing method showed higher salt scaling resistance than the method using the curing compound (except for the last five cycles).

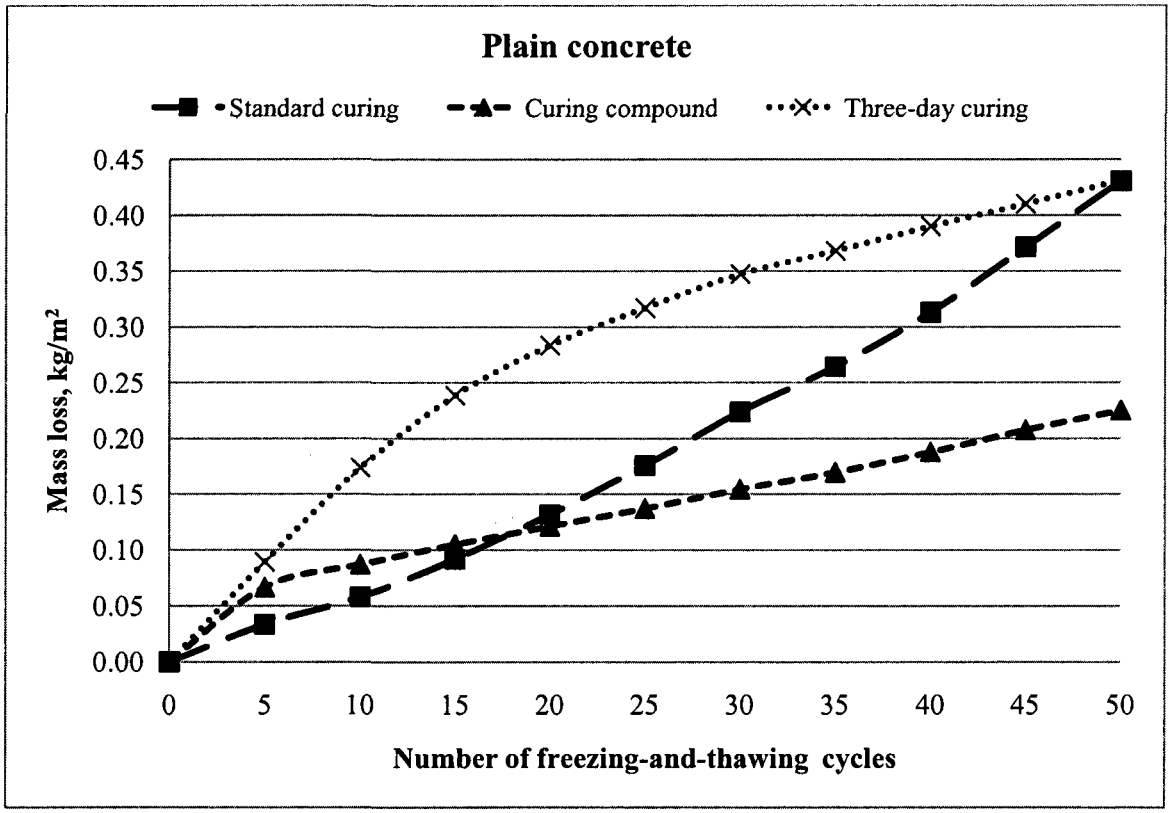

Figure 4.6 --- Mass loss comparison of different curing methods in the plain concrete.

As it can be seen in Figure 4.6, in the plain concrete made in this study, using the curing compound as specified in section 3.2.5 resulted in greater mass loss than the standard 14-day moist curing method until approximately the 20th cycle and less mass loss afterwards (48% less at the end of 50 freezing-and-thawing cycles).

On the other hand, the three-day moist curing method resulted in greater mass loss than the standard 14-day moist curing method throughout the 50 freezing-and-thawing cycles but the mass loss at the end of 50 cycles was exactly the same. It also can be observed that the three-day moist curing method showed greater mass loss than the method using the curing compound (92% more at the end of 50 freezing-and-thawing cycles).

Therefore, it can be concluded that in the plain concretes, the three-day moist curing method resulted in lower salt scaling resistance than both the standard 14-day moist curing method and the method using the curing compound. Based on the final mass loss at the end of 50 cycles, it can be concluded that the method using the curing compound showed higher salt scaling resistance than the standard 14-day moist curing method in spite of its lower salt scaling resistance until approximately the 20^{th} cycle.

As a summary, the method using the curing compound, as specified in section 3.2.5 with the mentioned dosage, showed lower salt scaling resistance than the standard 14-day moist curing method in the concretes made with fly ash and slag at 25% and 35% replacement of cement and higher salt scaling resistance in the plain concrete.

On the other hand, the three-day moist curing method showed higher salt scaling resistance than the standard 14-day moist curing method in the concrete made with fly ash but lower salt scaling resistance in the concrete made with slag and the plain concrete.

And finally the method using the curing compound showed lower salt scaling resistance than the three-day moist curing method in the concretes made with fly ash and slag but higher salt scaling resistance in the plain concrete.

4.4.3.3 Comparison of concrete types

In this section, all three concrete types made in this study including the plain concrete and the concretes made with 25% fly ash and 35% slag cement replacement will be compared in accordance to all variables studied in this research. These variables included ASTM C 672/C 672M (2003), its proposed replacement method, curing compound, and three-day curing. The relationship between mass loss and freezing-and-thawing cycles for all three concrete types according to each variable is presented in Figures 4.7 to 4.10.

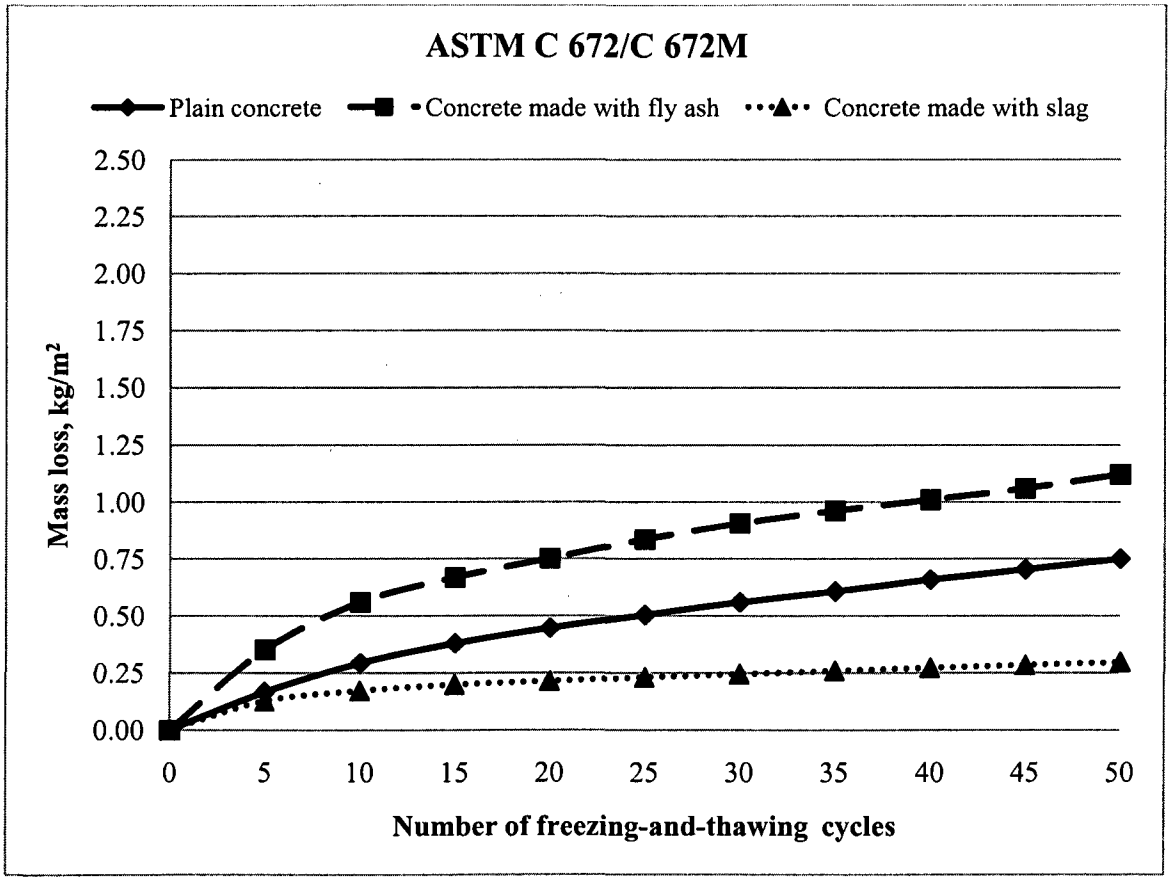

Figure 4.7 --- Mass loss comparison of different concrete types in accordance with ASTM C 672/C 672M (2003).

As it can be observed in Figure 4.7, among all three concrete types made in this study in accordance with ASTM C 672, the concrete made with fly ash showed the greatest mass loss (49% more than the plain concrete and 277% more than the concrete made with slag at the end of 50 cycles of freezing-and-thawing). The plain concrete also showed greater mass loss than the concrete made with slag (153% more at the end of 50 cycles of freezing-and-thawing).

Therefore it can be concluded that in accordance with ASTM C 672, the concrete made with slag showed the highest salt scaling resistance followed by the plain concrete followed by the concrete made with fly ash.

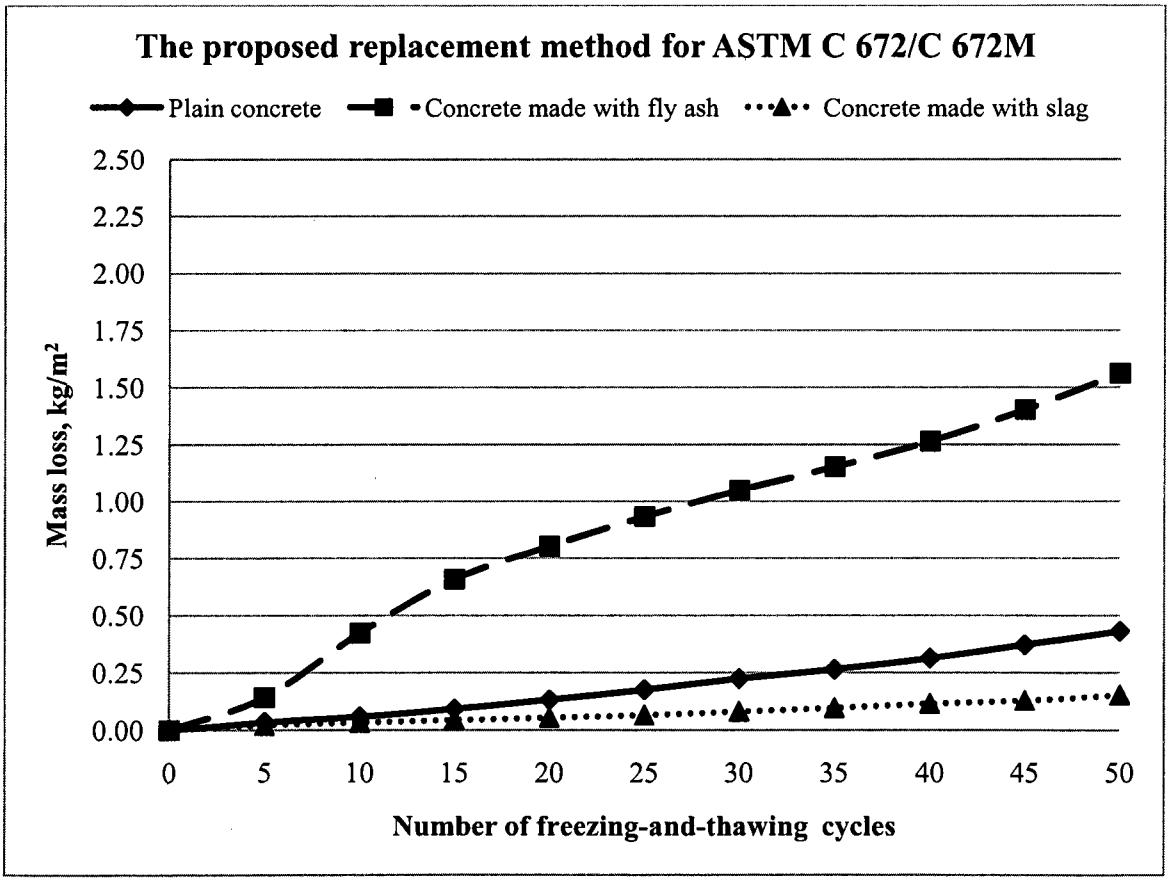

Figure 4.8 --- Mass loss comparison of different concrete types in accordance with the proposed replacement method for ASTM C 672/C 672M.

As it can be observed in Figure 4.8, among all three concrete types made in this study in accordance with the proposed replacement method for ASTM C 672/C 672M, the concrete made with fly ash showed the greatest mass loss (263% more than the plain concrete and 924% more than the concrete made with slag at the end of 50 cycles of freezing-and-thawing). The plain concrete also showed greater mass loss than the concrete made with slag (182% more at the end of 50 cycles of freezing-and-thawing).

Therefore it can be concluded that in accordance with the proposed replacement method for ASTM C 672/C 672M, the concrete made with slag showed the highest salt scaling resistance followed by the plain concrete followed by the concrete made with fly ash.

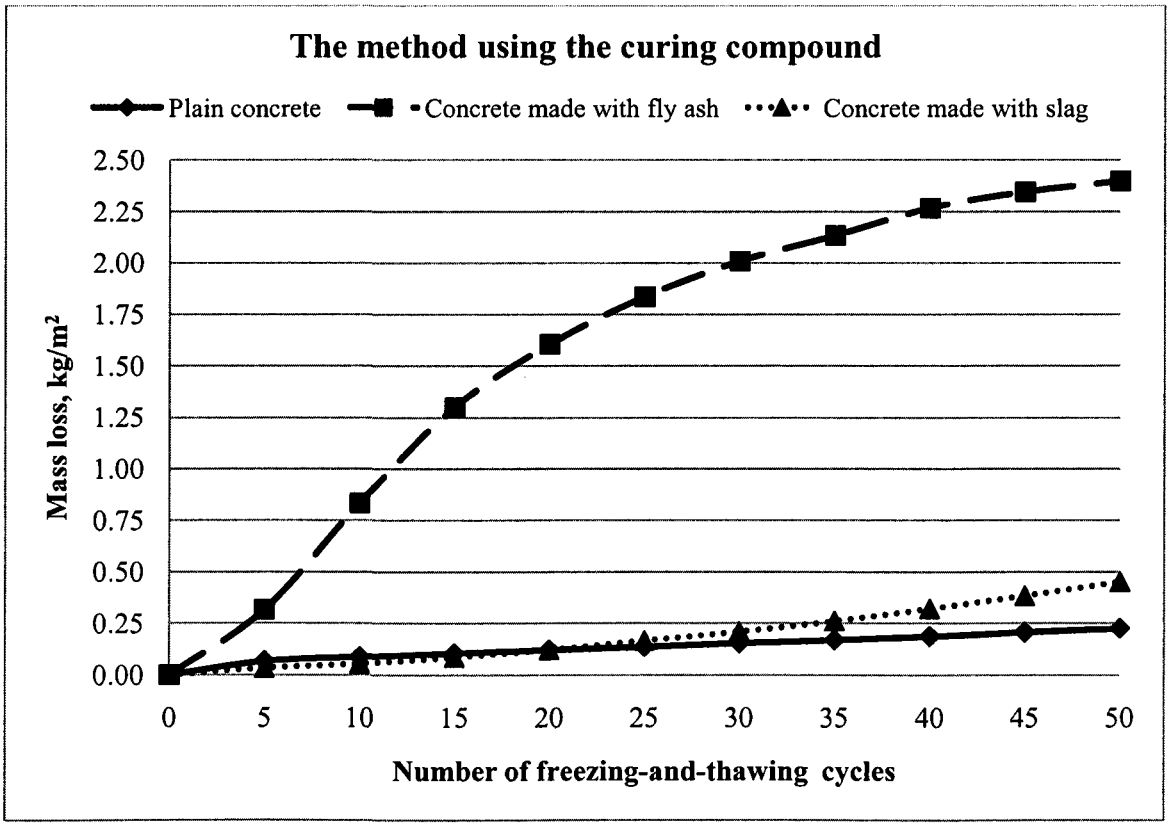

Figure 4.9 --- Mass loss comparison of different concrete types in accordance with the method using a curing compound.

As it can be observed in Figure 4.9, among all three concrete types made in this study in accordance with the method using the curing compound, the concrete made with fly ash showed the greatest mass loss (431% more than the concrete made with slag and 965% more than the plain concrete at the end of 50 cycles of freezing-and-thawing). The concrete made with slag also showed greater mass loss than the plain concrete (101% more at the end of 50 cycles of freezing-and-thawing) in spite of its slightly less mass loss before the 20^{th} cycle.

Therefore it can be concluded that in accordance with the method using the curing compound, the plain concrete showed the highest salt scaling resistance followed by the concrete made with slag followed by the concrete made with fly ash.

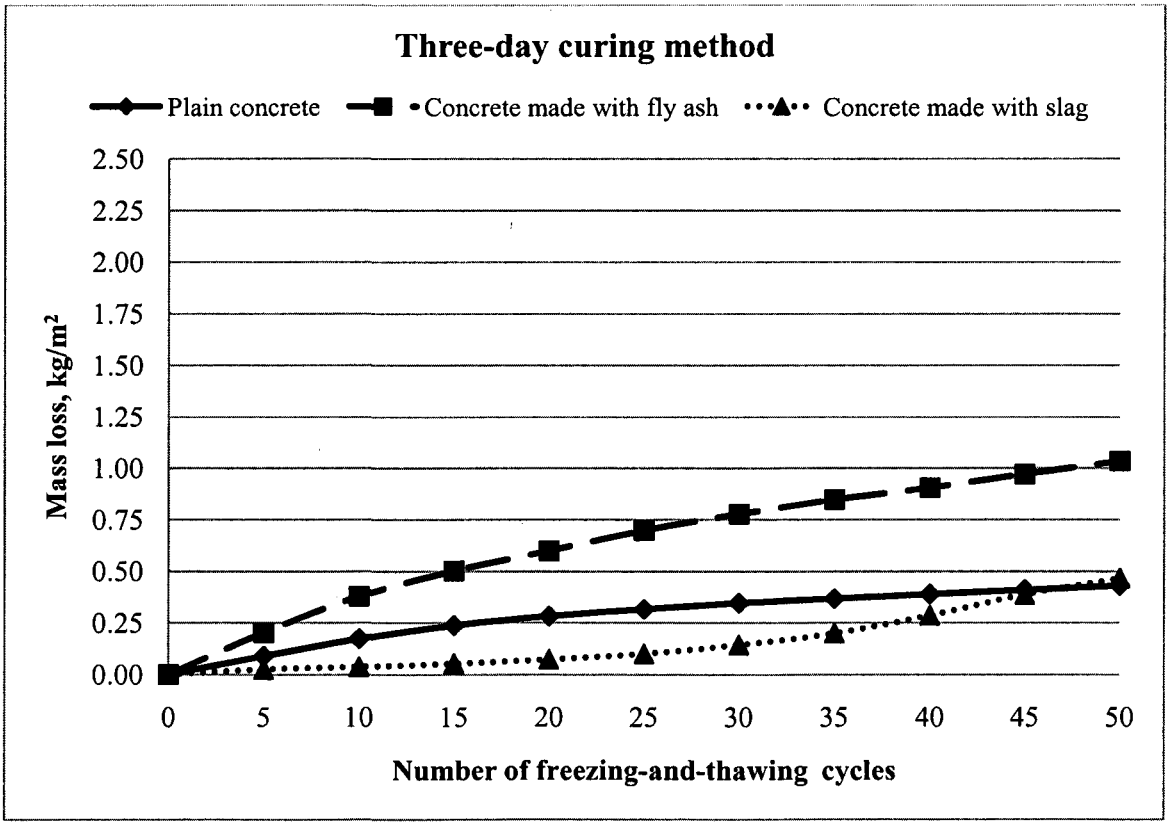

Figure 4.10 --- Mass loss comparison of different concrete types in accordance with the three-day curing method.

As it can be observed in Figure 4.10, among all three concrete types made in this study in accordance with the three-day curing method, the concrete made with fly ash showed the greatest mass loss (140% more than the plain concrete and 121% more than the concrete made with slag at the end of 50 cycles of freezing-and-thawing). The plain concrete also showed greater mass loss than the concrete made with slag throughout the freezing-and-thawing cycles until the 45^{th} cycle (5% more at the end of cycle #45) but slightly less mass loss at the end of 50 cycles (8% less).

Therefore it can be concluded that in accordance with the three-day curing method, the concrete made with slag showed the highest salt scaling resistance followed by the plain concrete followed by the concrete made with fly ash.

To summarize the findings, it can be concluded that in accordance to ASTM C 672, its proposed replacement method, and the three-day curing method, the concrete made with slag showed the highest salt scaling resistance followed by the plain concrete followed by the concrete made with fly ash. According to the method using the curing compound, the plain concrete showed the highest salt scaling resistance followed by the concrete made with slag followed by the concrete made with fly ash.

4.4.3.4 The additional scaling residue after detaching the plastic baseboard

After detaching the plastic baseboard from the scaling specimens at the end of 50 freezing-and-thawing cycles in the first concrete mixture, it was observed that some scaling residues were passed through the corners and attached to the plastic baseboard. Therefore, from the second concrete mixture, the additional scaling residue was collected and weighed to get an idea of its relationship to the cumulative scaling residue at the end

of 50 cycles. The additional scaling residue after detaching the plastic baseboard and its relationship with the scaling residue at the end of 50 freezing-and-thawing cycles for all scaling specimens are presented in Tables 4.7, 4.8, and 4.9.

Table 4.7 --- Scaling residue of the specimens made of the fly ash concrete.

			Cumulative scaling residue after 50 cycles, g	Additional scaling residue after detaching the plastic baseboard, g	Percentage of additional scaling residue to the cumulative scaling residue after 50 cycles, %
ASTM C 672	Standard 14-day moist curing	1st sample	55.709	N/A	N/A
		2nd sample	68.523	N/A	N/A
The proposed replacement method for ASTM C 672	Standard 14-day moist curing	1st sample	84.638	N/A	N/A
		2nd sample	88.356	N/A	N/A
	Curing compound	1st sample	141.754	22.096	15.59%
		2nd sample	123.955	14.631	11.80%
	Three-day curing	1st sample	63.165	1.985	3.14%
		2nd sample	51.425	0.852	1.66%

Table 4.8 --- Scaling residue of the specimens made of the slag concrete.

			Cumulative scaling residue after 50 cycles, g	Additional scaling residue after detaching the plastic baseboard, g	Percentage of additional scaling residue to the cumulative scaling residue after 50 cycles, %
ASTM C 672	Standard 14-day moist curing	1st sample	16.709	0.800	4.79%
		2nd sample	16.214	0.776	4.79%
The proposed replacement method for ASTM C 672	Standard 14-day moist curing	1st sample	8.312	0.810	9.74%
		2nd sample	8.581	1.033	12.04%
	Curing compound	1st sample	29.705	2.337	7.87%
		2nd sample	20.319	1.599	7.87%
	Three-day curing	1st sample	31.292	1.553	4.96%
		2nd sample	20.511	0.940	4.58%

Table 4.9 --- Scaling residue of the plain concrete specimens.

			Cumulative scaling residue after 50 cycles, g	Additional scaling residue after detaching the plastic baseboard, g	Percentage of additional scaling residue to the cumulative scaling residue after 50 cycles, %
ASTM C 672	Standard 14-day moist curing	1st sample	48.270	1.385	2.87%
		2nd sample	34.906	0.358	1.03%
The proposed replacement method for ASTM C 672	Standard 14-day moist curing	1st sample	22.328	0.473	2.12%
		2nd sample	25.383	0.620	2.44%
	Curing compound	1st sample	12.275	0.387	3.15%
		2nd sample	12.667	0.343	2.71%
	Three-day curing	1st sample	30.075	0.456	1.52%
		2nd sample	17.701	0.423	2.39%

This additional scaling residue after detaching the plastic baseboard cannot be clearly assigned to a specific cycle. The maximum ratio of this additional scaling residue to the cumulative scaling residue at the end of 50 freezing-and-thawing cycles was limited to 3.5%, 12%, and 15.5% in the plain concrete, the slag concrete, and the fly ash concrete, respectively.

Chapter 5

Discussion, conclusions, and recommendations

5.1 Introduction

The main emphasis of the experimental program presented in this research was to investigate the salt scaling resistance of three different concrete types using two different salt scaling test methods and three different curing regimes. In this chapter, the discussion and conclusions of this experimental research are presented. Also the recommendations for the further research in the area of salt scaling are given.

The main objectives of this experimental research were to compare the current ASTM C 672/C 672M standard (2003) with its proposed replacement method regarding the salt scaling resistance, to study the effect of different curing methods on the salt scaling resistance, and to compare the salt scaling resistance of different concrete types.

Three concrete types were made and tested according to both mentioned test methods including the plain concrete (100% Portland cement) and the concretes made with fly ash and slag at 25% and 35% replacement of cement, respectively. Three curing methods used in this research included the standard 14-day moist curing, three-day curing, and the usage of a curing compound.

In the previous chapter, the experimental results of the tests conducted on fresh concrete including slump, unit weight, and air content and the compressive strengths of all concrete mixtures prepared in this research were presented and discussed. Also the visual rating and the photographs of the scaled specimens at the end of 50 freezing-and-

thawing cycles according to both salt scaling test methods were given. The relationship between mass loss and the number of freezing-and-thawing cycles for all three concrete types made in this research in accordance with both salt scaling test methods and three different curing regimes were presented and discussed in detail in the previous chapter.

5.2 Discussion

The mass loss of all scaling specimens at the end of 50 freezing-and-thawing cycles for all three concrete types according to three curing regimes and two salt scaling test methods is presented in Table 5.1.

Table 5.1 --- Mass loss of all the scaling specimens at the end of 50 freezing-and-thawing cycles according to three curing methods.

		Cumulative mass loss after 50 cycles, kg/m^2					
		Concrete made with fly ash		Concrete made with Slag		Plain concrete	
		1st sample	2nd sample	1st sample	2nd sample	1st sample	2nd sample
ASTM C 672	Standard 14-day moist curing	1.006	1.237	0.302	0.293	0.871	0.630
The proposed replacement method for ASTM C 672	Standard 14-day moist curing	1.528	1.595	0.150	0.155	0.403	0.458
	Curing compound	2.559	2.237	0.536	0.367	0.222	0.229
	Three-day curing	1.140	0.928	0.565	0.370	0.543	0.320

It was observed in this research that the addition of fly ash to the concrete, at 25% cement replacement, decreased the salt scaling resistance according to all variables studied in this research (ASTM C 672, its proposed replacement method, the three-day curing method, and the method using the curing compound) in comparison with the other two concrete mixtures (slag concrete and the plain concrete). This reduction of salt scaling resistance when adding fly ash to the concrete has been addressed by several researchers (Gebler and Klieger, 1986; Johnston, 1987; Whiting, 1989; Bilodeau et al., 1991; Bilodeau and Malhotra, 1992; Afrani and Rogers, 1994; Bilodeau et al., 1994; Johnston, 1994; Boyd, 1995; Pigeon et al., 1996; Hooton and Boyd, 1997; Talbot et al., 2000; Bleszynski et al., 2002; Krishnan et al., 2006; Bouzoubaa et al., 2008). Part of the poor salt scaling resistance of the concrete made with fly ash was found to be related to its lower strength (46.1 MPa) in comparison with the plain concrete (54.6 MPa) or the concrete made with slag (51.7 MPa) which is in line with the findings of other researchers (Whiting, 1989; Bilodeau et al., 1991; Bilodeau and Malhotra, 1992). The concrete incorporating fly ash achieves higher strength at later curing periods (1-2 months) (Valenza II and Scherer, 2007a). Afrani and Rogers (1994) have addressed the early finishing (before bleeding has stopped due to the delayed set) and/or inadequate curing length as the reason of the lower salt scaling resistance of slag concrete which may be true in fly ash concrete too. They indicated that the former may cause a weak layer with higher w/c ratio to form near the surface and mentioned the reduced hydration heat at normal temperature as the cause for the latter.

On the other hand, substituting 35% of cement with slag increased the salt scaling resistance in accordance with ASTM C 672 and its proposed replacement method while it

decreased the resistance according to the method using the curing compound significantly and the three-day curing method slightly. The scaling specimens made of slag concrete when tested according to the three-day curing method showed lower scaling residue than the plain concrete specimens until the 45th cycle but slightly higher scaling residue was observed at the end of 50 freezing-and-thawing cycles. This could be due the higher air content of slag concrete (7.0%) in comparison with the plain concrete (5.0 or 5.5%) or fly ash concrete (5.5 or 6.5%). Bouzoubaa et al. (2008) observed a higher salt scaling resistance of the concrete made with 25% slag than the control concrete when tested in accordance with ASTM C 672 and addressed the higher air content as the reason but they indicated that the concrete made with 35% slag performed poorly (moderate to severe scaling, level 4) in accordance with ASTM C 672. In contrast, some other researchers reported a reduction in salt scaling resistance when slag was added to the concrete (Bilodeau and Malhotra, 1993; Setzer, 1993; Copuroglu et al., 2004).

It was also observed in this research that the concrete made with slag showed greater salt scaling resistance than the concrete made with fly ash regardless of the curing methods or the salt scaling test methods. This could be due to the higher compressive strength and air content of slag concrete (51.7 MPa and 7.0%, respectively) in comparison with fly ash concrete (46.1 MPa and 5.5 or 6.5%, respectively). Bouzoubaa et al. (2008) also reported the higher salt scaling resistance of slag concrete in comparison with fly ash concrete and addressed the higher air content and lower air-void spacing factor as the reason.

The proposed replacement method resulted in greater salt scaling resistance than ASTM C 672 in the plain concrete and the concrete made with slag. Bouzoubaa et al.

(2008) reported a poor performance of the concrete made with 35% slag (moderate to severe scaling, level 4) in accordance with ASTM C 672 while a satisfactory performance (level 1) according to BNQ procedure (scaled significantly less). The proposed replacement method for ASTM C 672 is based on the method developed by Bureau de Normalisation du Quebec (BNQ). The better salt scaling resistance of the proposed replacement is thought to be due to one-week pre-saturation period which may balance the ions between the saline solution and the concrete's voids, decrease the osmotic pressure and result in less salt scaling (Bouzoubaa et al., 2008). Bouzoubaa et al. (2008) found the ASTM C 672 more severe in the laboratory than the field (as agreed to by Hooton and Boyd, 1997; Thomas, 1997; Langley and Leaman, 1998; Bleszynski et al., 2002; Marchand et al., 2005; Krishnan et al., 2006; Boyd and Hooton, 2007) and addressed the BNQ procedure as a better way to evaluate the salt scaling resistance of concrete incorporating supplementary cementing materials. They reported greater scaling residue and higher visual rating of specimens tested in accordance with ASTM C 672 than the BNQ standard in concrete specimens made with fly ash and slag.

On the other hand, ASTM C 672 showed higher salt scaling resistance than its proposed replacement method in the concrete made with fly ash based on the final mass loss at the end of 50 freezing-and-thawing cycles, in spite of its lower resistance until approximately the 15^{th} cycle. Although the proposed replacement method started out with less mass loss than ASTM C 672, the rate of increase was high throughout and resulted in a greater mass loss at the end of 50 freezing-and-thawing cycles. This is in contrast with the other two concrete mixtures made in this study or the published data and further research in this area is recommended. Bouzoubaa et al. (2008) observed poor salt scaling

resistance (severe scaling, level 5) of concrete mixtures incorporating fly ash at 25% and 35% cement replacement in accordance with ASTM C 672 but a satisfactory resistance (level 2 for 35% fly ash and level 3 for 25%) according to BNQ (2002) in comparison with the control mixture (level 1 according to both ASTM C 672 and BNQ) which is in line with other published data (Gebler and Klieger, 1986; Johnston, 1987; Whiting, 1989; Bilodeau et al., 1991; Johnston, 1994; Hooton and Boyd, 1997; Talbot et al., 2000). They also reported greater scaling residue and higher visual rating of specimens tested in accordance with ASTM C 672 than the BNQ standard in all control mixture and fly ash concretes. Bouzoubaa et al. (2008) used 3% NaCl solution in both ASTM C 672 and the BNQ standard while in this research 4% calcium chloride ($CaCl_2$) was used in both ASTM C 672 and its proposed replacement method which could be the reason of different results in these two studies. Another possible reason for the lower salt scaling of the specimens made with fly ash concrete could be the excessive finishing of the concrete surface since it was the first concrete mixture made in this study and it is possible that the finishing was done too early before the bleeding has stopped. However, all four specimens were cast from the same batch and the source of this poor performance remains in doubt. Bouzoubaa et al. (2008) mentioned that by brushing the surface too early or much later (after the bleeding has stopped), the surface air-void network may be damaged which results in lower salt scaling resistance. Bilodeau et al. (1994) also indicated that finishing may change the air-void system, making it insufficient to resist the freezing-and-thawing damage, resulting in poor scaling performance in concrete mixtures containing high volume of fly ash. However, they used 55 to 60 % fly ash as the cement replacement which is much higher than the amount used in this research.

The application of the particular curing compound as used in this research resulted in different results for all three mixtures investigated. Table 5.2 presents the ranking for salt scaling resistance of three different curing methods in each concrete type made in this study.

Table 5.2 --- Salt scaling resistance ranking of three different curing methods in all three concrete types.

	25% fly ash	35% slag	Plain concrete
Standard 14-day moist curing	2	1	2
Curing compound	3	2	1
Three-day curing	1	3	2

Note: 1 indicates the highest salt scaling resistance, 3 the lowest

In the concrete made with fly ash, the curing compound resulted in significantly lower salt scaling resistance than the standard 14-day moist curing method, while the three-day moist curing method showed higher salt scaling resistance than both the standard 14-day moist curing method and the method using the curing compound. On the other hand, in the concrete made with slag, using the curing compound as well as the three-day moist curing method resulted in lower salt scaling resistance than the standard 14-day moist curing method. For this mixture, the mass loss at 50 cycles was nearly the same for the curing compound and three-day curing. Finally, in the plain concretes, the curing compound showed significantly more salt scaling resistance than the two other curing methods investigated. The three-day moist curing method showed similar salt scaling

resistance to the standard 14-day moist curing method. Based on the final mass loss at the end of 50 freezing-and-thawing cycles, higher salt scaling resistance of the method using the curing compound was observed in comparison with the standard 14-day moist curing method in spite of its lower salt scaling resistance until approximately the 20^{th} cycle.

The application of the curing compound in this study resulted in lower salt scaling resistance than the standard 14-day moist curing method in the concretes made with fly ash and slag at 25% and 35% replacement of cement, respectively and higher salt scaling resistance in the plain concrete. In this research, after applying the curing compound on the concrete surface, the samples were kept in the laboratory room temperature until the age of 28 days when the one-week pre-saturation started. The surface of the specimens was not brushed off by a metal brush or other tools and therefore the weak layer at the top was not removed. This could be the reason of the poor salt scaling resistance of fly ash and slag concretes. It has been reported that using the sealers or curing compounds (surface treatment) postpones the surface scaling (Afrani and Rogers, 1993; Sedran et al., 1993; Setzer, 1993; Cleland and Basheer, 2002) but has no influence on the salt scaling resistance at the end (Perenchio, 1988; Setzer, 1993; Cleland and Basheer, 2002). Boyd (1995) indicated that the pores in the surface of fresh concrete may be blocked when the curing compound is applied which stops the moisture penetration and leads to a "false" good frost resistance. Afrani and Rogers (1994) indicated that applying the curing compound showed a good salt scaling resistance at the early stage according to ASTM C 672 but after five cycles, the rate of the scaling was very fast.

On the other hand, Pigeon et al. (1987) reported an improved salt scaling resistance in concrete made with silica fume and the application of a curing compound where the

surface of the samples was brushed with a metallic mechanized brush to remove as much as the weak layer having the curing compound prior to testing. Boyd and Hooton (2007) also showed a better salt scaling resistance in the specimens treated with the curing compound in comparison with the specimens covered under burlap and plastic sheet and Gebler and Klieger (1986) indicated that in concrete mixtures containing 25% fly ash, a slightly better salt scaling resistance of the method using the curing compound (stored at 23 °C) was observed than the air curing and the moist curing at this temperature. Radlinski et al. (2008) reported the higher salt scaling resistance of the specimens treated with the curing compound exposed both at the early and late age in comparison with air drying (storage at 23 °C and 50% relative humidity), three and seven days under wet burlap and covered with plastic sheet which is in line with the findings by Krishnan et al. (2006). Radlinski et al. (2008) suggested that a thin layer of curing compound avoids the salt and moisture penetration; reduce the saturation degree of the pore system in the very top layer, and results in the good performance at the early age in these specimens.

The three-day moist curing method showed higher salt scaling resistance than the standard 14-day moist curing method in the concrete made with fly ash but lower salt scaling resistance in the concrete made with slag. At the end of 50 freezing-and-thawing cycles, the same salt scaling resistance of the plain concrete specimens was observed in both the three-day moist curing method and the standard 14-day moist curing method in spite of the lower resistance of three-day curing method throughout the salt scaling test. Bilodeau et al. (1991, 1998) indicated that in fly ash concrete, extending the moist curing period in the laboratory (from 28 to 91 days) decreased the salt scaling resistance, which could be the reason for better salt scaling resistance of fly ash concrete in this research

cured under the three-day curing method in comparison with other two curing methods. Moist curing was reported to increase the scaling resistance of the concrete incorporating the class F fly ash having reached moisture stabilization prior to the salt scaling test (Waktola et al., 2005). On the other hand, the improvement of salt scaling resistance due to the longer moist curing periods was reported by some researchers (Langlois et al., 1989; Gagne et al., 1991) which could be the reason for better salt scaling resistance of the standard 14-day moist curing method in the plain concrete and the concrete made with slag in this research.

And finally, using the curing compound resulted in lower salt scaling resistance than the three-day moist curing method in the concretes made with fly ash, slightly higher resistance in slag concrete, and significantly higher resistance in the plain concrete.

5.3 Conclusions

Based on the relationship between mass loss and the number of freezing-and-thawing cycles for all three concrete types made in this research in accordance with both salt scaling test methods and three different curing regimes presented in the Chapter 4, the following conclusions can be drawn from the present experimental research:

1. Compared to the other concrete mixtures, the addition of fly ash to the concrete, at 25% cement replacement, decreased the salt scaling resistance according to all variables studied in this research (ASTM C 672, its proposed replacement method, the three-day curing method, and the method using the curing compound). This could be due to the lower compressive strength of the fly ash concrete, early finishing, and/or inadequate curing length.

2. Compared to the other concrete mixtures, the addition of slag to the concrete, at 35% cement replacement, increased the salt scaling resistance in accordance with ASTM C 672 and its proposed replacement method while it decreased the resistance according to the method using the curing compound significantly and the three-day curing method slightly. This could be due to the higher air content of slag concrete in comparison with the plain concrete or fly ash concrete.
3. The specimens tested according the proposed replacement method showed greater salt scaling resistance than ASTM C 672 in the plain concrete and the concrete made with slag at 35% replacement of cement. This is likely due to one-week pre-saturation period which may balance the ions between the saline solution and the concrete's voids, decrease the osmotic pressure and result in less salt scaling. This result agrees with the published literature.
4. In the concrete made with fly ash, based on the final mass loss at the end of 50 freezing-and-thawing cycles, ASTM C 672 showed higher salt scaling resistance than its proposed replacement method in spite of its lower resistance until approximately the 15^{th} cycle. This is in contrast with the other two concrete mixtures made in this study or the published data and further research in this area is recommended. This could be to the application of different de-icing salt, excessive finishing, and/or early finishing before the bleeding has stopped. However, all four specimens were cast from the same batch.
5. The application of the curing compound in this study resulted in lower salt scaling resistance than the standard 14-day moist curing method in the concretes made with fly ash and slag at 25% and 35% replacement of cement, respectively, and higher salt

scaling resistance in the plain concrete. This could be because of not removing the weak layer from the surface of fly ash and slag specimens by using a metal brush or other tools. However, in the plain concrete this weak layer seemed to be not as critical as in fly ash and slag concretes. Using the curing compound resulted in lower salt scaling resistance in fly ash concrete, slightly higher resistance in slag concrete and significantly higher resistance in the plain concrete than the three-day moist curing method (based on the final mass loss at the end of 50 freezing-and-thawing cycles).

6. The three-day moist curing method showed higher salt scaling resistance than the standard 14-day moist curing method in the concrete made with fly ash, lower salt scaling resistance in the concrete made with slag, and similar salt scaling resistance in the plain concrete (based on the final mass loss at the end of 50 freezing-and-thawing cycles). This could be due to the longer moist curing period which was found by other researchers to have a negative effect on the salt scaling resistance of fly ash concrete and the positive effect in the plain concrete and the concrete made with slag.
7. The concrete made with slag showed the highest salt scaling resistance in accordance with ASTM C 672 and its proposed replacement method followed by the plain concrete followed by the concrete made with fly ash. This could be due the higher air content of slag concrete in comparison with the other concrete mixtures. According to the method using the curing compound and the three-day curing method, the highest salt scaling resistance was observed in the plain concrete followed by the concrete made with slag, followed by the concrete made with fly ash (based on the final mass loss at the end of 50 freezing-and-thawing cycles). However, the mass loss of the scaling specimens made of slag concrete at the end of 50 freezing-and-thawing cycles

was slightly higher than the plain concrete specimens according to the three-day curing method while this difference was really significant in accordance with the method using the curing compound.

5.4 Recommendations for further research

To study the effect of other parameters on salt scaling resistance of a concrete surface and to provide a comprehensive understanding of the salt scaling, its characteristics and mechanisms further research is needed. Some of the recommended investigations which should be noticed in the further researches are as follows:

1. The scaling specimens made of fly ash concrete showed lower salt scaling resistance when tested according to the proposed replacement method in comparison with ASTM C 672 which was in contrast with the other two concrete mixtures made in this study (slag concrete or the plain concrete) or the published papers. Therefore, further research is recommended regarding the comparison of ASTM C 672 with its replacement method in fly ash concrete from the point of view of salt scaling resistance to verify the validity of the findings of this experimental research.
2. Application of all other curing methods mentioned in Chapter 2 at the same time for each concrete mixture is recommended to study the effect of curing on salt scaling resistance and to find the most appropriate curing method resulting in the highest resistance.
3. The investigation regarding the effect of the curing compound on salt scaling resistance could be continued by using other dosages of the curing compound and

studying the effect of brushing the concrete surface prior to freezing-and-thawing cycles (removing the weak layer).

4. The effect of silica fume and other replacement amounts of fly ash and slag on salt scaling resistance according to these two salt scaling test methods could be investigated.
5. Application of other de-icing salts rather than calcium chloride ($CaCl_2$) or other concentrations could be studied.
6. Although compressive strength was only used for information purposes, equipment improvements such as the ability of maintaining the load rate control is recommended.

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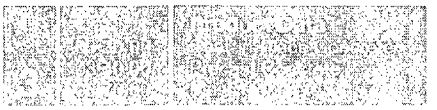
Appendix A

Chemical admixtures, product information

BASF

The Chemical Company



POLYHEED® 997

Mid-Range Water-Reducing Admixture

Description

PolyHeed 997 admixture is a patented multi-component, non-chloride, mid-range water-reducing admixture. PolyHeed 997 admixture meets ASTM C 494/C 494M requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications

Recommended for use in:

- Conventionally placed concrete mixes containing a wide range of cements, slag cement, Class C and F fly ashes, silica fume and aggregates
- Reinforced, precast, prestressed, lightweight or normal weight concrete and pumped concrete
- Residential/commercial flatwork and formed surfaces
- Rheodynamic® Self-Consolidating Concrete
- 4x4™ Concrete
- Pervious Concrete

Features

- True mid-range water reduction (5-15%) and excellent performance across a wide concrete slump range, especially the difficult slump range of 5-8 in. (125-200 mm)
- Superior workability, pumpability and finishability qualities even in concrete mixes containing low amounts of cementitious materials
- Compressive and flexural strength performance increased at all ages
- Strength performance comparable to chloride-bearing, water-reducing admixtures at all ages
- Superior finishing characteristics for residential/commercial flatwork and formed surfaces

Benefits

- Significantly reduced placement and finishing time resulting in lower in-place concrete costs
- Improved concrete durability to damage from freezing and thawing
- Increased service life of concrete structures

Performance Characteristics

Mixture Data: 500 lb/yd^3 (295 kg/m^3) of Type I cement; slump, 6-7 in. (150-180 mm); 5-6% air; concrete temperature 70 °F (21 °C); ambient temperature, 70 °F (21 ° C).

Setting Time Performance[1]

Mixture	Initial Set h:min	Difference h:min
Plain	6:01	–
PolyHeed 997 admixture @		
5 fl oz/cwt (325 mL/100 kg)	6:22	+0:21
10 fl oz/cwt (650 mL/100 kg)	6:57	+0:56
15 fl oz/cwt (980 mL/100 kg)	7:31	+1:30

Compressive Strength Performance

Mixture	7-Day psi	7-Day MPa	7-Day %	28-Day psi	28-Day MPa	28-Day %
Plain	2360	16.3	100	3320	22.9	100
PolyHeed 997 admixture @						
5 fl oz/cwt (325 mL/100 kg)	3060	21.1	129	3930	27.1	118
10 fl oz/cwt (650 mL/100 kg)	3740	25.8	158	4610	31.8	136
15 fl oz/cwt (980 mL/100 kg)	4620	31.9	196	5460	37.7	165

[1] *Note: The data shown are based on controlled laboratory tests. Reasonable variations from the results shown here may be experienced as a result of differences in concrete making materials and jobsite conditions.*

Master Builders

Product Data: POLYHEED® 997

Guidelines for Use

Dosage: PolyHeed 997 admixture has a recommended dosage range of 3-15 fl oz /cwt (195-980 mL/100 kg) of cementitious material for most concrete mixes.

As the dosage of PolyHeed 997 admixture increases to 15 fl oz/cwt (980 mL/100 kg) of cementitious materials, normal concrete setting characteristics are maintained and early and ultimate compressive strengths increase.

BASF Construction Chemicals does not recommend the use of dosages outside the recommended range without trial testing. Consult your local BASF Construction Chemicals sales representative for assistance in determining the dosage for optimum performance.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: PolyHeed 997 admixture will neither initiate nor promote corrosion of reinforcing or prestressing steel embedded in concrete, or of galvanized steel floor and roof systems. PolyHeed 997 admixture does not contain intentionally added calcium chloride or other chloride-based ingredients.

Compatibility: PolyHeed 997 admixture may be used in combination with any BASF Construction Chemicals admixtures. When used in conjunction with other admixtures, each admixture must be dispensed separately into the concrete mixture.

Storage and Handling

Storage Temperature: If PolyHeed 997 admixture freezes, thaw at 35 °F (2 °C) or above and completely reconstitute by mild mechanical agitation. ***Do not use pressurized air for agitation.***

Shelf Life: PolyHeed 997 admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of PolyHeed 997 admixture has been exceeded.

Packaging

PolyHeed 997 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: PolyHeed 997 admixture.

Additional Information

For additional information on PolyHeed 997 admixture or its use in developing concrete mixtures with special performance characteristics, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

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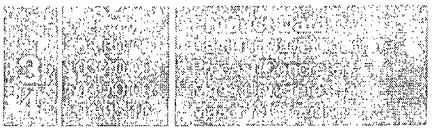
The Chemical Company



GLENIUM® 7500

High-Range Water-Reducing Admixture

Description

GLENIUM® 7500 high-range water-reducing admixture is based on the next generation of polycarboxylate technology found in all of the Glenium 7000 series products. This technology combines state-of-the-art molecular engineering with a precise understanding of regional cements to provide specific and exceptional value to all phases of the concrete construction process.

GLENIUM 7500 admixture is very effective in producing concrete mixtures with different levels of workability including applications that require self-consolidating concrete (SCC). The use of GLENIUM 7500 admixture results in faster setting characteristics as well as improved early age compressive strength. GLENIUM 7500 admixture meets ASTM C 494/C 494M compliance requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications

Recommended for use in:

- Concrete with varying water reduction requirements (5-40%)
- Concrete where control of workability and setting time is critical
- Concrete where high flowability, increased stability, high early and ultimate strengths, and improved durability are needed
- Production of Rheodynamic® Self-Consolidating Concrete (SCC) mixtures
- 4x4™ Concrete for fast-track construction
- Pervious Concrete mixtures

Features

- Excellent early strength development
- Controls setting characteristics
- Optimizes slump retention/setting relationship
- Consistent air entrainment
- Dosage flexibility

Benefits

- Faster turnover of forms due to accelerated early strength development
- Reduces finishing labor costs due to optimized set times
- Use in fast track construction
- Minimizes the need for slump adjustments at the jobsite
- Less jobsite QC support required
- Fewer rejected loads
- *Optimizes concrete mixture costs*

Performance Characteristics

Concrete produced with GLENIUM 7500 admixture achieves significantly higher early age strength than first generation polycarboxylate high-range water-reducing admixtures. GLENIUM 7500 admixture also strikes the perfect balance between workability retention and setting characteristics in order to provide efficiency in placing and finishing concrete.

Guidelines for Use

Dosage: GLENIUM 7500 admixture has a recommended dosage range of 2-15 fl oz/cwt (130-975 mL/100 kg) of cementitious materials. For most applications, dosages in the range of 5-8 fl oz/cwt (325-520 mL/100 kg) will provide excellent performance. For high performance and Rheodynamic Self-Consolidating Concrete mixtures, dosages of up to 12 fl oz/cwt (780 mL/100 kg) of cementitious materials can be utilized. Because of variations in concrete materials, jobsite conditions and/or applications, dosages outside of the recommended range may be required. In such cases, contact your local BASF Construction Chemicals representative.

Mixing: GLENIUM 7500 admixture can be added with the initial batch water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Master Builders

Product Data: GLENIUM® 7500

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: GLENIUM 7500 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of GLENIUM 7500 admixture.

Compatibility: GLENIUM 7500 admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid-range and high-range water-reducing admixtures, air-entrainers, accelerators, retarders, extended set control admixtures, corrosion inhibitors, and shrinkage reducers.

Do not use GLENIUM 7500 admixture with admixtures containing beta-naphthalene sulfonate. Erratic behaviors in slump, workability retention and pumpability may be experienced.

Storage and Handling

Storage Temperature: GLENIUM 7500 admixture must be stored at temperatures above 40 °F (5 °C). If GLENIUM 7500 admixture freezes, thaw and reconstitute by mechanical agitation.

Shelf Life: GLENIUM 7500 admixture has a minimum shelf life of 6 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of GLENIUM 7500 admixture has been exceeded.

Packaging

GLENIUM 7500 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: GLENIUM 7500 admixture.

Additional Information

For additional information on GLENIUM 7500 admixture or on its use in developing concrete mixtures with special performance characteristics, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative admixtures for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the North American region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

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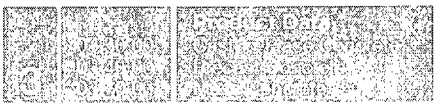

Description

MB-AE 90 air-entraining admixture is for use in concrete mixtures. It meets the requirements of ASTM C 260, AASHTO M 154 and CRD-C 13.

Applications

Recommended for use in:

- Concrete exposed to cyclic freezing and thawing
- Production of high-quality normal or lightweight concrete (heavyweight concrete normally does not contain entrained air)

MB-AE™ 90

Air-Entraining Admixture

Features

- Ready-to-use in the proper concentration for rapid, accurate dispensing

Benefits

- Improved resistance to damage from cyclic freezing and thawing
- Improved resistance to scaling from deicing salts
- Improved plasticity and workability
- Reduced permeability – increased watertightness
- Reduced segregation and bleeding

Performance Characteristics

Concrete durability research has established that the best protection for concrete from the adverse effects of freezing and thawing cycles and deicing salts results from: proper air content in the hardened concrete, a suitable air-void system in terms of bubble size and spacing, and adequate concrete strength, assuming the use of sound aggregates and proper mixing, transporting, placing, consolidation, finishing and curing techniques. MB-AE 90 admixture can be used to obtain adequate freeze-thaw durability in a properly proportioned concrete mixture, if standard industry practices are followed.

Air Content Determination: The total air content of normal weight concrete should be measured in strict accordance with ASTM C 231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method" or ASTM C 173/C 173M, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method." The air content of lightweight concrete should only be determined using the Volumetric Method. The air content should be verified by calculating the gravimetric air content in accordance with ASTM C 138/C 138M, "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete." If the total air content, as measured by the Pressure Method or Volumetric Method and as verified by the Gravimetric Method, deviates by more than 1-1/2%, the cause should be determined and corrected through equipment calibration or by whatever process is deemed necessary.

Guidelines for Use

Dosage: There is no standard dosage for MB-AE 90 admixture. The exact quantity of air-entraining admixture needed for a given air content of concrete varies because of differences in concrete-making materials and ambient conditions. Typical factors that might influence the amount of air entrained include: temperature, cementitious materials, sand gradation, sand-aggregate ratio, mixture proportions, slump, means of conveying and placement, consolidation and finishing technique.

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Product Data: MB-AE™ 90

The amount of MB-AE 90 admixture used will depend upon the amount of entrained air required under actual job conditions. In a trial mixture, use 1/4 to 4 fl oz/cwt (16-260 mL/100 kg) of cementitious material. Measure the air content of the trial mixture, and, if needed, either increase or decrease the quantity of MB-AE 90 admixture to obtain the desired air content.

In mixtures containing water-reducing or set-control admixtures, the amount of MB-AE 90 admixture needed may be somewhat less than the amount required in plain concrete.

Due to possible changes in the factors that can affect the dosage of MB-AE 90 admixture, frequent air content checks should be made during the course of the work. Adjustments to the dosage should be based on the amount of entrained air required in the mixture at the point of placement.

If an unusually high or low dosage of MB-AE 90 admixture is required to obtain the desired air content, consult your BASF Construction Chemicals representative. In such cases, it may be necessary to determine that, in addition to a proper air content in the fresh concrete, a suitable air-void system is achieved in the hardened concrete.

Dispensing and Mixing: Add MB-AE 90 admixture to the concrete mixture using a dispenser designed for air-entraining admixtures, or add manually using a suitable measuring device that ensures accuracy within plus or minus 3% of the required amount.

For optimum, consistent performance, the air-entraining admixture should be dispensed on damp, fine aggregate. If the concrete mixture contains fine lightweight aggregate, field evaluations should be conducted to determine the best method to dispense the air-entraining admixture.

Precaution

In a 2005 publication from the Portland Cement Association (PCA R&D Serial No. 2789), it was reported that problematic air-void clustering that can potentially lead to above normal decreases in strength was found to coincide with late additions of water to air-entrained concretes. Late additions of water include the conventional practice of holding back water during batching for addition at the jobsite. Therefore, caution should be exercised with delayed additions of water to air-entrained concrete. Furthermore, an air content check should be performed after any post-batching addition to an air-entrained concrete mixture.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: MB-AE 90 admixture will neither initiate nor promote corrosion of reinforcing and prestressing steel embedded in concrete, or of galvanized floor and roof systems. No calcium chloride or other chloride-based ingredients are used in the manufacture of this admixture.

Compatibility: MB-AE 90 admixture may be used in combination with any BASF Construction Chemicals admixture, unless stated otherwise on the data sheet for the other product. When used in conjunction with other admixtures, each admixture must be dispensed separately into the concrete mixture.

Storage and Handling

Storage Temperature: MB-AE 90 admixture should be stored and dispensed at 31 °F (-0.5 °C) or higher. Although freezing does not harm this product, precautions should be taken to protect it from freezing. If MB-AE 90 admixture freezes, thaw at 35 °F (2 °C) or above and completely reconstitute by mild mechanical agitation. ***Do not use pressurized air for agitation.***

Shelf Life: MB-AE 90 admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of MB-AE 90 admixture has been exceeded.

Safety: Chemical goggles and gloves are recommended when transferring or handling this material.

Packaging

MB-AE 90 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: MB-AE 90 admixture.

Additional Information

For additional information on MB-AE 90 admixture, or its use in developing a concrete mixture with special peformance characteristics, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

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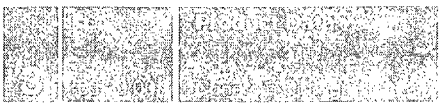

Description

Confilm evaporation reducer helps produce high quality concrete flatwork and reduces surface moisture evaporation. Because Confilm retards evaporation, it is especially effective in combating rapid drying conditions including, high concrete and/or ambient temperatures, low humidity, high winds, direct sunlight or work in heated interiors during cold weather, etc.

Applications

Recommended for use in:

- Concrete surfaces where the evaporation rate exceeds the rate of bleeding of the concrete
- Air-entrained and non-air-entrained concrete
- Silica fume concrete
- Concrete containing fly ash

CONFILM®

Evaporation Reducer

Features

- Reduces surface moisture evaporation about 80% in wind and about 40% in sunlight
- Eliminates or reduces crusting, stickiness and underlying sponginess which often cause unevenness and poor surface texture
- Reduces and, in many instances, eliminates plastic shrinkage cracking and wind crusting of flatwork surfaces
- Supplements the recommended practices for hot weather concreting
- Safeguards against the ill effects of evaporation
- Allows lower slump and lower unit water content in concrete used for flatwork
- Facilitates use of air-entrained concrete, required for durability and workability, in situations where air-entrainment might be avoided for fear that it would increase concrete's susceptibility to crusting and stickiness under drying conditions

Benefits

- Virtually eliminates need to add extra mixing water to compensate for rapid evaporation during finishing
- Saves time and money by increasing the amount of surface handled per finisher even under rapid drying conditions
- Timing of the operations is less critical, thus reducing overall cost

Performance Characteristics

A detailed technical discussion about the action of monomolecular films, typified by Confilm evaporation reducer, is contained in the Journal of the American Concrete Institute, Volume 62, pp. 977-985. The use of a monomolecular film to prevent rapid drying of fresh concrete is recommended in the following ACI documents: ACI 302.1R, "Guide for Concrete Floor and Slab Construction"; ACI 305R, "Hot Weather Concreting"; ACI 308R, "Guide to Curing Concrete"; and ACI 345R, "Guide for Concrete Highway Bridge Deck Construction".

Guidelines for Use

Dosage: One gal (3.8 L) of Confilm evaporation reducer mixed with 9 gal (34.1 L) of water yields 10 gal (37.9 L) of sprayable solution. This diluted amount of Confilm evaporation reducer (1:9) should cover 2,000 to 4,000 ft^2 (186 to 372 m^2) of fresh concrete. If more than one application of Confilm is made, as under adverse drying conditions, the quantity required will be increased accordingly.

Mixing: Depending on the application, Confilm evaporation reducer can be mixed at a ratio of up to 1 part Confilm concentrate to 9 parts of water. Agitate Confilm evaporation reducer before mixing with water. Re-agitate mixed materials before applying.

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Product Data: CONFILM®

Application: Apply with a constant pressure or industrial-type sprayer. Confilm evaporation reducer forms a monomolecular film when sprayed onto the concrete surface immediately after screeding and/or between finishing operations (as needed). This protective shield usually lasts as long as concrete remains plastic, despite succeeding floating and troweling operations.

Treated surfaces are easily distinguished from untreated surfaces because of the greenish-yellow color of the film in the presence of surface moisture and ultraviolet rays (sunlight or artificial lighting). The fluorescent color tint disappears completely upon drying. The residue remaining on the surface of hardened concrete does not impair bonding or alter color.

Product Notes

Precaution: Do not use Confilm evaporation reducer as a finishing aid to facilitate finishing of:

- Cementitious dry-shake surface hardeners or toppings
- Air-entrained and non-air-entrained concrete, silica fume concrete or fly ash concrete

Confilm evaporation reducer is not to be applied during final flatwork steel troweling operations. Confilm evaporation reducer is not a curing agent. Concrete treated with this product must still be cured. BASF Construction Chemicals is not responsible for compatibility or results when Confilm evaporation reducer is used with other manufacturers' products. Confilm reduces evaporation only while concrete is in its plastic state. It is not a substitute for early curing of hardened concrete nor does it alter the effectiveness of membrane-type curing compounds. Confilm evaporation reducer is not to be applied during any finishing operation nor should it be worked into the concrete surface.

Any residue remaining from spillage or spraying of Confilm evaporation reducer concentrate on the surface of hardened concrete should not be allowed to dry. Wipe it immediately, then rinse the surface with water. If the Confilm concentrate residue is allowed to dry on hardened concrete, a reddish-brown stain may appear. To remove the stain, place a cloth saturated in a household-type, chlorinated bleach onto the stain, then cover it with plastic to retard evaporation. After approximately one hour, the stain should disappear completely. Rinse the area with water.

Storage and Handling

Storage Temperature: Confilm evaporation reducer must be protected from freezing. Extreme cold may cause segregation after which the product cannot be reconstituted.

Shelf Life: Confilm evaporation reducer has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use if the shelf life of Confilm evaporation reducer has been exceeded.

Packaging

Confilm evaporation reducer is supplied in 1, 5 and 55 gal (3.8, 18.9 and 208 L) containers.

Related Documents

Material Safety Data Sheets: Confilm evaporation reducer.

Additional Information

For suggested specification information or for additional product data on Confilm evaporation reducer, contact your BASF Construction Chemicals representative.

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Appendix B

Details of the tests results

B.1 Tests conducted on the aggregates

In this study, the tests conducted on the aggregates included sieve analysis, specific gravity, absorption, bulk density, and the moisture content.

B.1.1 Sieve analyses

Sieve analyses were conducted on both the fine and coarse aggregates used in this research in accordance with ASTM C 136 (2006) and are given in Figures 3.1 and 3.2, respectively. The detailed information is given in Tables B.1 and B.2. The initial mass of the fine and coarse aggregates was 350 and 600 grams, respectively. The duration of sieve analysis test was 15 minutes.

Table B.1 --- Sieve analysis of the fine aggregate used in this study.

Sieve size	Mass, g			Percentage of retained aggregate on each sieve, %	Cumulative percentage of retained aggregate, %	Percentage of passed aggregate from each sieve, %
	Empty sieve	Sieve and aggregate	Retained aggregate			
9.51 mm (3/8 in)	558.3	560.0	1.7	0.49	0.49	99.51
4.76 mm (No. 4)	489.9	511.8	21.9	6.31	6.80	93.20
2.38 mm (No.8)	489.3	520.1	30.8	8.88	15.68	84.32
1.19 mm (No.16)	454.2	498.6	44.4	12.80	28.47	71.53
0.60 mm (No.30)	422.8	484.1	61.3	17.67	46.14	53.86
0.30 mm (No.50)	378.1	469.8	91.7	26.43	72.56	27.44
0.15 mm (No.100)	360.2	434.4	74.1	21.35	93.92	6.08
Pan	322.7	343.8	21.1	6.08	100.00	0.00
Total	-	-	347.0	100.00	-	-

Table B.2 --- Sieve analysis of the coarse aggregate used in this study.

Sieve size	Mass, g			Percentage of retained aggregate on each sieve, %	Cumulative percentage of retained aggregate, %	Percentage of passed aggregate from each sieve, %
	Empty sieve	Sieve and aggregate	Retained aggregate			
37.5 mm (1 ½ in)	566.7	566.7	0.0	0.00	0.00	100.00
31.5 mm (1 ¼ in)	558.8	558.8	0.0	0.00	0.00	100.00
25.4 mm (1 in)	529.0	529.0	0.0	0.00	0.00	100.00
19 mm (¾ in)	595.8	595.8	0.0	0.00	0.00	100.00
12.5 mm (½ in)	569.7	569.7	0.0	0.00	0.00	100.00
9.5 mm (3/8 in)	593.2	634.8	41.6	6.99	6.99	93.01
4.76 mm (No.4)	522.5	999.6	477.1	80.14	87.13	12.87
2.36 mm (No.8)	488.6	559.0	70.4	11.83	98.96	1.04
Pan	378.0	384.2	6.2	1.04	100.00	0.00
Total	-	-	595.3	100.00	-	-

B.1.2 Specific gravity

Specific gravity of both the fine and coarse aggregates was determined according to ASTM C 128 (2007) and ASTM C 127 (2007), respectively and is given in Chapter 3, Table 3.3. The detailed information is given in Tables B.4 and B.5.

Table B.3 --- Preparation information of specific gravity of the fine aggregate.

	Mass, g
Oven-dried aggregate (A)	506.83
506.83 g of oven-dried fine aggregate is equal to 510 g of SSD fine aggregate (S)	
Pycnometer, aggregate, and water (C)	1012.6
Pycnometer and water (B)	692.06

The specific gravity of the fine aggregate was calculated according to ASTM C 128 (2007) as follows:

Specific gravity (oven dry) = A / (B+S-C), (B-1)

Specific gravity (saturated-surface-dry) = S / (B+S-C), (B-2)

Specific gravity (apparent dry) = A / (B+A-C), (B-3)

where:

A = mass of oven-dry specimen = 506.83 g,

B = mass of pycnometer filled with water to calibration mark, B = 692.06 g,

C = mass of pycnometer filled with specimen and water to calibration mark,

C = 1012.60 g,

S = mass of saturated-surface-dry specimen, S = 510.00 g.

Table B.4 --- Preparation information of specific gravity of the coarse aggregate.

	Mass, g
Oven-dried aggregate (A)	2095.0
Saturated-surface-dry aggregate (B)	2067.7
Basket in water	850.0
Basket and aggregate in water	2100.0
Aggregate in water (C)	1250.0

The specific gravity of the coarse aggregate was calculated according to ASTM C 127 (2007) as follows:

Specific gravity (oven dry) = A / (B-C), (B-4)

Specific gravity (saturated-surface-dry) = B / (B-C), (B-5)

Specific gravity (apparent dry) = A / (A-C), (B-6)

where:

A = mass of oven-dry test sample in air, A = 2095.0 g,

B = mass of saturated-surface-dry test sample in air, B = 2067.7 g,

C = apparent mass of saturated test sample in water, C = 1250.0 g.

B.1.3 Absorption

Absorption of the fine and coarse aggregates was determined in accordance with ASTM C 128 (2007) and ASTM C 127 (2007), respectively and is given in Chapter 3, Section 3.3.3. The detailed information is given in Tables B.5 and B.6. Two 200 litre barrels of the coarse aggregate were used in this research. Barrel number 2 was used in concrete mixture numbers one to four while barrel number one was used in the last two concrete mixtures (numbers five and six).

Table B.5 --- Preparation information of absorption of the fine aggregate.

	Mass, g
Saturated-surface-dry aggregate (S)	445.63
Oven-dried aggregate after being cooled in the room temperature (A)	442.86

The absorption of the fine aggregate was calculated according to ASTM C 128 (2007) as follows:

Absorption, % = 100 [(S-A) / A], (B-7)

where:

A = mass of oven-dry specimen = 442.86 g,

S = mass of saturated-surface-dry specimen, S = 445.63 g.

Table B.6 --- Preparation information of absorption of the coarse aggregate.

	Mass, g			
	Barrel #1		Barrel #2	
	1st test	2nd test	1st test	2nd test
Saturated-surface-dry aggregate (B)	1963.80	2423.90	1990.20	2276.50
Oven-dried aggregate after being cooled in the room temperature (A)	1943.77	2398.16	1972.64	2253.53

The absorption of the coarse aggregate was calculated according to ASTM C 127 (2007) as follows:

Absorption, % = 100 [(B-A) / A], (B-8)

where:

A = mass of oven-dry test sample in air, g,

B = mass of saturated-surface-dry test sample in air, g.

B.1.4 Bulk density

Bulk density of both the fine and coarse aggregates was determined according to ASTM C 29/C 29M (2007) and is given in Chapter 3, Section 3.3.4. The detailed information is given in Table B.7.

Table B.7 --- Preparation information of bulk density of the aggregates.

	Mass, g	
	Fine aggregate	Coarse aggregate
Measure alone (T)	3.94	
Measure full of water	11.10	
Measure full of aggregate (G)	15.50	14.47
Density of water, kg/m^3	1000	
Volume of measure, m^3 (V)	0.00716	

The bulk density of the aggregate was calculated according to C 29/C 29M (2007) as follows:

$$M = (G-T) / V, \quad \text{(B-9)}$$

where:

M = bulk density of the aggregate, kg/m^3,
G = mass of the aggregate plus the measure, kg,
T = mass of the measure, T = 3.94 kg,
V = volume of the measure, V = 0.00716 m^3.

B.1.5 Moisture content

In this study, the moisture content of both the fine and coarse aggregates was determined in accordance with ASTM C 566 (2004) and is given in Chapter 3, Table 3.5. The original and the adjusted amount of water needed for each concrete mixture are given in Chapter 3, Table 3.6. The detailed information is given in Tables B.8 and B.9.

Table B.8 --- Preparation information of moisture content of the fine aggregate.

	Mass, g					
	Mix #1	Mix #2	Mix #3	Mix #4	Mix #5	Mix #6
Aggregate in the room temperature (W)	1000.50	1000.30	1000.70	1000.40	1000.13	1000.87
Oven-dried aggregate (D)	984.60	977.30	977.30	992.84	962.34	946.58

Table B.9 --- Preparation information of moisture content of the coarse aggregate.

	Mass, g					
	Mix #1	Mix #2	Mix #3	Mix #4	Mix #5	Mix #6
Aggregate in the room temperature (W)	1000.70	1000.90	1000.90	1000.60	1000.95	1000.74
Oven-dried aggregate (D)	997.70	994.00	994.00	997.66	994.18	993.89

The moisture content of the aggregate was calculated according to ASTM C 566 (2004) as follows:

$$P = 100\,[(W-D) / D], \tag{B-10}$$

where:

P = total evaporable moisture content of sample, percent,

W = mass of original sample, g,

D = mass of dried sample, g.

The adjusted amount of water needed for each concrete mixture was calculated as following and is given in Chapter 3:

$$\text{Water needed} = \text{original water} \pm \text{water in the fine and coarse aggregates} \tag{B-11}$$

$$= \text{original water} + [\text{mass of aggregate} * (\text{water absorption} - \text{moisture Content})]$$

B.2 Tests conducted on fresh concrete

Tests conducted on fresh concrete in this research included the slump according to ASTM C 143/C 143M (2008), the unit weight in accordance with ASTM C 138/C 138M (2008), and the air content according to ASTM C 231 (2008). The experimental results of these tests are given in Chapter 4, Table 4.1 and the detailed information of the unit weight is given in Table B.10.

Table B.10 --- Preparation information of the unit weight of each concrete mixture.

		Mass, kg			Density of water, kg/m^3	Unit weight of concrete, kg/m^3
		Empty measure (M_m)	Measure filled with concrete (M_c)	Measure filled with water (M_w)		
Concretes made with fly	Mix #1	3.93	19.97	11.10	1,000	2,237
	Mix #2	3.93	20.19	11.10	1,000	2,268
Concretes made with slag	Mix #3	3.93	20.11	11.10	1,000	2,257
	Mix #4	3.93	19.98	11.10	1,000	2,238
Plain concrete	Mix #5	3.93	20.62	11.10	1,000	2,328
	Mix #6	3.93	20.39	11.10	1,000	2,296

The unit weight of the concrete was calculated according to ASTM C 138/C 138M (2008) as follows:

$$D = (M_c - M_m) / V_m, \quad \text{(B-12)}$$

where:

D = unit weight (density), kg/m^3,
M_c = mass of the measure filled with the concrete, kg,
M_m = mass of the measure, kg,
M_w = mass of the measure filled with water, kg,
V_m = volume of the measure, m^3,
= $(M_w - M_m)$ / density of water.

B.3 Compression test

The compression test was conducted on all three cylinders of each concrete mixture made in this study according to ASTM C 39/C 39M (2005) and the results are given in Chapter 4, Table 4.2. The detailed information is given in Table B.11 and the calculation details are described in Chapter 4, Section 4.3.

Table B.11 --- Information of the compression test of each concrete mixture.

			Failure Load		Compressive strength, MPa
		Sample #	lbf	N	
Concrete made with fly ash	Mix #1	1	87,100	387,440	49.3
		2	82,900	368,758	47.0
		3	82,400	366,533	46.7
	Mix #2	1	80,700	358,972	45.7
		2	78,200	347,851	44.3
		3	77,100	342,958	43.7
Concrete made with slag	Mix #3	1	96,700	430,143	54.8
		2	97,700	434,591	55.3
		3	98,100	436,371	55.6
	Mix #4	1	84,900	377,654	48.1
		2	84,800	377,209	48.0
		3	85,300	379,433	48.3
Plain concrete	Mix #5	1	104,400	464,394	59.1
		2	104,100	463,060	59.0
		3	102,600	456,388	58.1
	Mix #6	1	88,000	391,444	49.8
		2	91,000	404,788	51.5
		3	88,400	393,223	50.1

B.4 Salt scaling test

Salt scaling test was conducted according to ASTM C 672/C 672M (2003) and its proposed replacement method using three different curing methods (standard 14-day moist curing, three-day curing, and the usage of a curing compound) and the results are given in Chapter 4, Figures 4.5 to 4.14. The detailed information is given in Tables B.12 to B.26.

Table B.12 --- Date of casting and salt scaling test of the concrete mixtures made with fly ash.

			Casting	Freezing-and-thawing cycles	
				Start	Finish
Mix #1	ASTM C 672	Standard 14-day moist curing	October 23rd, 2008	November 20th, 2008	January 9th, 2009
	The proposed replacement method			November 27th, 2008	January 16th, 2009
Mix #2	The proposed replacement method	Curing compound	November 26th, 2008	December 31st, 2008	February 19th, 2009
		Three-day curing		December 31st, 2008	February 19th, 2009

Table B.13 --- Date of casting and salt scaling test of the concrete mixtures made with slag.

			Casting	Freezing-and-thawing cycles	
				Start	Finish
Mix #3	ASTM C 672	Standard 14-day moist curing	December 12th, 2008	January 9th, 2009	February 28th, 2009
	The proposed replacement method			January 16th, 2009	March 7th, 2009
Mix #4	The proposed replacement method	Curing compound	January 16th, 2009	February 20th, 2009	April 12th, 2009
		Three-day curing		February 20th, 2009	April 12th, 2009

Table B.14 --- Date of casting and salt scaling test of the plain concrete.

			Casting	Freezing-and-thawing cycles	
				Start	Finish
Mix #5	ASTM C 672	Standard 14-day moist curing	February 20th, 2009	March 20th, 2009	May 9th, 2009
	The proposed replacement method			March 27th, 2009	May 17th, 2009
Mix #6	The proposed replacement method	Curing compound	March 23rd, 2009	April 27th, 2009	June 16th, 2009
		Three-day curing		April 27th, 2009	June 16th, 2009

Table B.15 --- Scaling residue of each specimen made with fly ash concrete (1st 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				5	10	15	20	25
Mix #1	ASTM C 672	Standard 14-day moist curing	1	22.305	32.454	37.038	40.368	43.702
			2	16.547	29.480	37.036	42.982	48.877
			Average	19.426	30.967	37.037	41.675	46.290
	Proposed replacement		1	10.384	27.095	39.127	47.019	54.311
			2	5.440	19.776	33.805	41.967	49.132
			Average	7.912	23.436	36.466	44.493	51.722
Mix #2	The proposed replacement method for ASTM C 672	Curing compound	1	20.135	50.983	79.650	97.357	111.583
			2	14.953	41.474	64.319	80.471	91.905
			Average	17.544	46.229	71.985	88.914	101.744
		Three-day curing	1	11.228	21.921	29.446	35.712	41.962
			2	11.282	20.053	26.238	30.666	35.421
			Average	11.255	20.987	27.842	33.189	38.692

Table B.16 --- Scaling residue of each specimen made with fly ash concrete (2nd 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				30	35	40	45	50
Mix #1	ASTM C 672	Standard 14-day moist curing	1	47.006	49.261	51.320	53.245	55.709
			2	53.500	57.415	60.665	64.256	68.523
			Average	50.253	53.338	55.993	58.751	62.116
	Proposed replacement		1	60.329	65.220	70.502	76.914	84.638
			2	55.915	62.444	69.640	78.558	88.356
			Average	58.122	63.832	70.071	77.736	86.497
Mix #2	The proposed replacement method for ASTM C 672	Curing compound	1	122.488	129.618	137.452	139.870	141.754
			2	100.120	106.828	113.779	120.064	123.955
			Average	111.304	118.223	125.616	129.967	132.855
		Three-day curing	1	47.321	51.361	54.695	58.954	63.165
			2	38.841	42.541	45.591	48.685	51.425
			Average	43.081	46.951	50.143	53.820	57.295

Table B.17 --- Scaling residue of each specimen made with slag concrete (1st 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				5	10	15	20	25
Mix #3	ASTM C 672	Standard 14-day moist curing	1	7.528	10.072	11.475	12.476	13.339
			2	6.507	8.944	10.534	11.235	12.146
			Average	7.018	9.508	11.005	11.856	12.743
	Proposed replacement		1	0.984	1.673	2.163	2.673	3.316
			2	1.273	2.033	2.693	3.328	3.948
			Average	1.129	1.853	2.428	3.001	3.632
Mix #4	The proposed replacement method for ASTM C 672	Curing compound	1	1.987	3.351	5.319	7.480	10.265
			2	1.755	2.727	4.199	6.050	8.180
			Average	1.871	3.039	4.759	6.765	9.223
		Three-day curing	1	1.489	2.210	3.097	4.367	6.053
			2	1.323	1.985	2.792	3.793	5.150
			Average	1.406	2.098	2.945	4.080	5.602

Table B.18 --- Scaling residue of each specimen made with slag concrete (2nd 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				30	35	40	45	50
Mix #3	ASTM C 672	Standard 14-day moist curing	1	14.209	14.906	15.510	16.044	16.709
			2	12.912	13.821	14.677	15.467	16.214
			Average	13.561	14.364	15.094	15.756	16.462
	Proposed replacement		1	4.281	5.180	6.252	6.948	8.312
			2	4.699	5.524	6.544	7.335	8.581
			Average	4.490	5.352	6.398	7.142	8.447
Mix #4	The proposed replacement method for ASTM C 672	Curing compound	1	12.671	15.999	20.015	24.587	29.705
			2	10.573	12.956	15.616	17.997	20.319
			Average	11.622	14.478	17.816	21.292	25.012
		Three-day curing	1	8.802	12.173	18.888	25.602	31.292
			2	7.226	10.084	12.980	17.500	20.511
			Average	8.014	11.129	15.934	21.551	25.902

Table B.19 --- Scaling residue of each specimen made with plain concrete (1st 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				5	10	15	20	25
Mix #5	ASTM C 672	Standard 14-day moist curing	1	10.701	19.271	24.718	29.143	32.729
			2	7.585	13.114	17.367	20.328	23.111
			Average	9.143	16.193	21.043	24.736	27.920
	Proposed replacement		1	1.839	3.133	4.987	7.304	9.593
			2	1.875	3.305	5.197	7.310	9.892
			Average	1.857	3.219	5.092	7.307	9.743
Mix #6	The proposed replacement method for ASTM C 672	Curing compound	1	3.872	5.138	6.144	6.967	7.723
			2	3.529	4.529	5.499	6.476	7.489
			Average	3.701	4.834	5.822	6.722	7.606
		Three-day curing	1	6.486	13.016	17.566	20.639	22.745
			2	3.425	6.222	8.874	10.756	12.369
			Average	4.956	9.619	13.220	15.698	17.557

Table B.20 --- Scaling residue of each specimen made with plain concrete (2nd 25 cycles).

			Sample #	Cumulative mass of scaling residue after cycle #, g				
				30	35	40	45	50
Mix #5	ASTM C 672	Standard 14-day moist curing	1	36.240	39.258	42.388	45.263	48.270
			2	25.801	28.074	30.731	32.828	34.906
			Average	31.021	33.666	36.560	39.046	41.588
	Proposed replacement		1	12.063	13.930	16.362	19.321	22.328
			2	12.747	15.346	18.304	21.847	25.383
			Average	12.405	14.638	17.333	20.584	23.856
Mix #6	The proposed replacement method for ASTM C 672	Curing compound	1	8.638	9.391	10.279	11.406	12.275
			2	8.482	9.392	10.533	11.612	12.667
			Average	8.560	9.392	10.406	11.509	12.471
		Three-day curing	1	24.814	26.198	27.619	28.857	30.075
			2	13.660	14.581	15.608	16.598	17.701
			Average	19.237	20.390	21.614	22.728	23.888

Table B.21 --- Mass loss of each specimen made with fly ash concrete (1st 25 cycles).

			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				5	10	15	20	25
Mix #1	ASTM C 672	Standard 14-day moist curing	1	0.403	0.586	0.669	0.729	0.789
			2	0.299	0.532	0.669	0.776	0.882
			Average	0.351	0.559	0.669	0.752	0.836
	Proposed replacement		1	0.187	0.489	0.706	0.849	0.980
			2	0.098	0.357	0.610	0.758	0.887
			Average	0.143	0.423	0.658	0.803	0.934
Mix #2	The proposed replacement method for ASTM C 672	Curing compound	1	0.363	0.920	1.438	1.757	2.014
			2	0.270	0.749	1.161	1.453	1.659
			Average	0.317	0.834	1.299	1.605	1.837
		Three-day curing	1	0.203	0.396	0.532	0.645	0.757
			2	0.204	0.362	0.474	0.554	0.639
			Average	0.203	0.379	0.503	0.599	0.698

Table B.22 --- Mass loss of each specimen made with fly ash concrete (2nd 25 cycles).

			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				30	35	40	45	50
Mix #1	ASTM C 672	Standard 14-day moist curing	1	0.848	0.889	0.926	0.961	1.006
			2	0.966	1.036	1.095	1.160	1.237
			Average	0.907	0.963	1.011	1.060	1.121
	Proposed replacement		1	1.089	1.177	1.273	1.388	1.528
			2	1.009	1.127	1.257	1.418	1.595
			Average	1.049	1.152	1.265	1.403	1.561
Mix #2	The proposed replacement method for ASTM C 672	Curing compound	1	2.211	2.340	2.481	2.525	2.559
			2	1.807	1.928	2.054	2.167	2.237
			Average	2.009	2.134	2.267	2.346	2.398
		Three-day curing	1	0.854	0.927	0.987	1.064	1.140
			2	0.701	0.768	0.823	0.879	0.928
			Average	0.778	0.847	0.905	0.971	1.034

Table B.23 --- Mass loss of each specimen made with slag concrete (1st 25 cycles).

			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				5	10	15	20	25
Mix #3	ASTM C 672	Standard 14-day moist curing	1	0.136	0.182	0.207	0.225	0.241
			2	0.117	0.161	0.190	0.203	0.219
			Average	0.127	0.172	0.199	0.214	0.230
	Proposed replacement		1	0.018	0.030	0.039	0.048	0.060
			2	0.023	0.037	0.049	0.060	0.071
			Average	0.020	0.033	0.044	0.054	0.066
Mix #4	The proposed replacement method for ASTM C 672	Curing compound	1	0.036	0.060	0.096	0.135	0.185
			2	0.032	0.049	0.076	0.109	0.148
			Average	0.034	0.055	0.086	0.122	0.166
		Three-day curing	1	0.027	0.040	0.056	0.079	0.109
			2	0.024	0.036	0.050	0.068	0.093
			Average	0.025	0.038	0.053	0.074	0.101

Table B.24 --- Mass loss of each specimen made with slag concrete (2nd 25 cycles).

			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				30	35	40	45	50
Mix #3	ASTM C 672	Standard 14-day moist curing	1	0.256	0.269	0.280	0.290	0.302
			2	0.233	0.249	0.265	0.279	0.293
			Average	0.245	0.259	0.272	0.284	0.297
	Proposed replacement		1	0.077	0.094	0.113	0.125	0.150
			2	0.085	0.100	0.118	0.132	0.155
			Average	0.081	0.097	0.115	0.129	0.152
Mix #4	The proposed replacement method for ASTM C 672	Curing compound	1	0.229	0.289	0.361	0.444	0.536
			2	0.191	0.234	0.282	0.325	0.367
			Average	0.210	0.261	0.322	0.384	0.451
		Three-day curing	1	0.159	0.220	0.341	0.462	0.565
			2	0.130	0.182	0.234	0.316	0.370
			Average	0.145	0.201	0.288	0.389	0.468

Table B.25 --- Mass loss of each specimen made with plain concrete (1st 25 cycles).

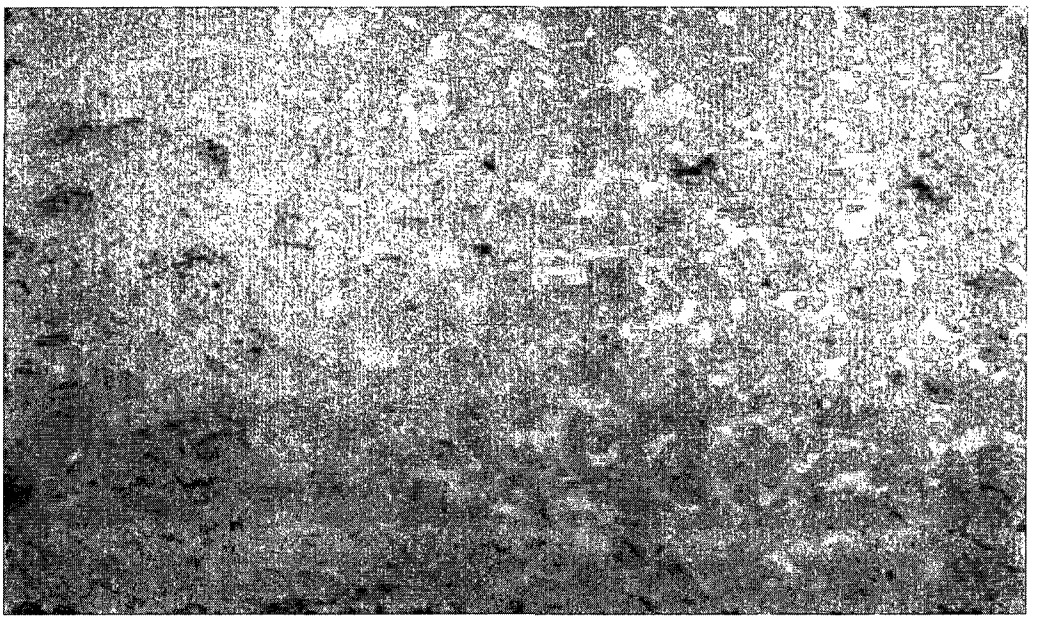
			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				5	10	15	20	25
Mix #5	ASTM C 672	Standard 14-day moist curing	1	0.193	0.348	0.446	0.526	0.591
			2	0.137	0.237	0.313	0.367	0.417
			Average	0.165	0.292	0.380	0.446	0.504
	Proposed replacement		1	0.033	0.057	0.090	0.132	0.173
			2	0.034	0.060	0.094	0.132	0.179
			Average	0.034	0.058	0.092	0.132	0.176
Mix #6	The proposed replacement method for ASTM C 672	Curing compound	1	0.070	0.093	0.111	0.126	0.139
			2	0.064	0.082	0.099	0.117	0.135
			Average	0.067	0.087	0.105	0.121	0.137
		Three-day curing	1	0.117	0.235	0.317	0.373	0.411
			2	0.062	0.112	0.160	0.194	0.223
			Average	0.089	0.174	0.239	0.283	0.317

Table B.26 --- Mass loss of each specimen made with plain concrete (2nd 25 cycles).

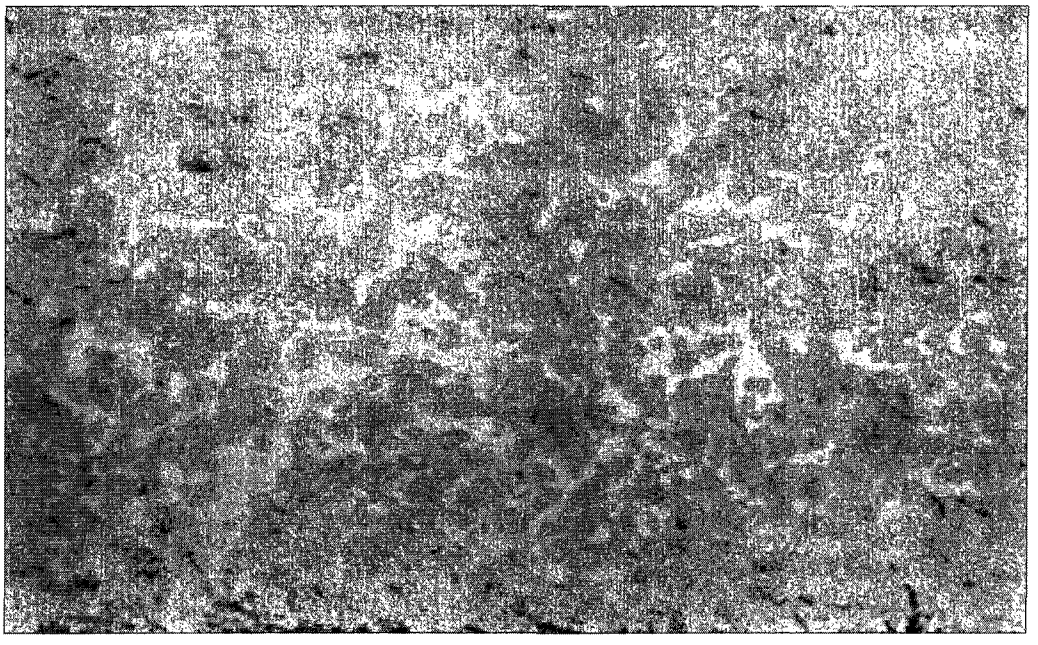
			Sample #	Cumulative mass loss after cycle #, kg/m^2				
				30	35	40	45	50
Mix #5	ASTM C 672	Standard 14-day moist curing	1	0.654	0.709	0.765	0.817	0.871
			2	0.466	0.507	0.555	0.593	0.630
			Average	0.560	0.608	0.660	0.705	0.751
	Proposed replacement		1	0.218	0.251	0.295	0.349	0.403
			2	0.230	0.277	0.330	0.394	0.458
			Average	0.224	0.264	0.313	0.372	0.431
Mix #6	The proposed replacement method for ASTM C 672	Curing compound	1	0.156	0.170	0.186	0.206	0.222
			2	0.153	0.170	0.190	0.210	0.229
			Average	0.155	0.170	0.188	0.208	0.225
		Three-day curing	1	0.448	0.473	0.499	0.521	0.543
			2	0.247	0.263	0.282	0.300	0.320
			Average	0.347	0.368	0.390	0.410	0.431

Appendix C

Photographs of scaling specimens after 50 freezing-and-thawing cycles

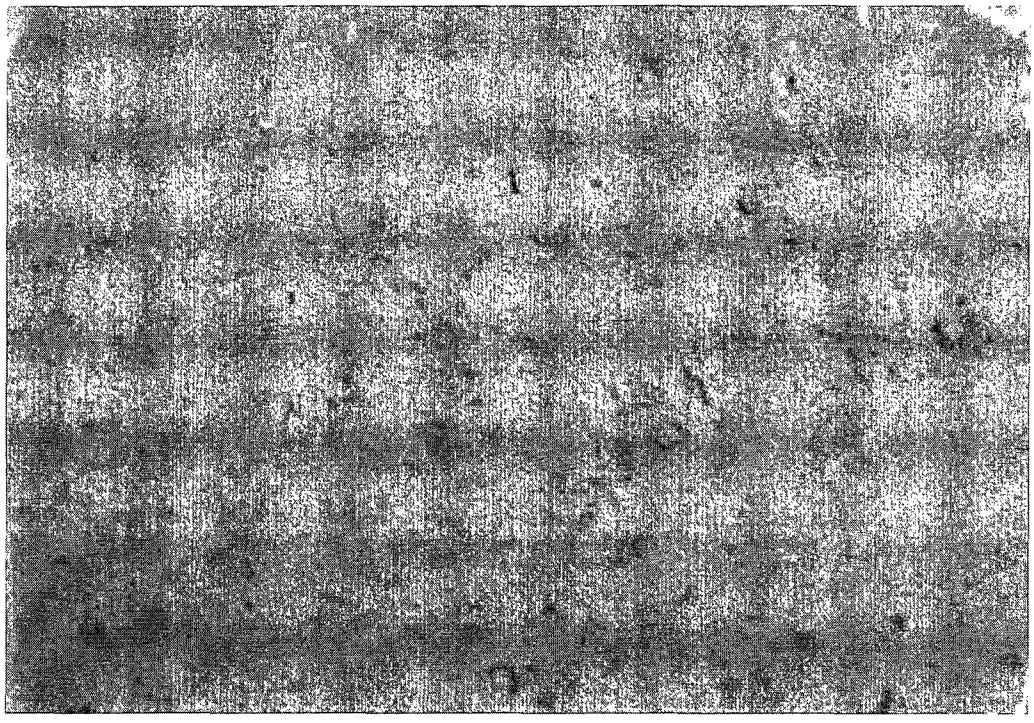


1st sample



2nd sample

Figure C.1 — Photos of the scaling specimens made of fly ash concrete according to ASTM C 672.



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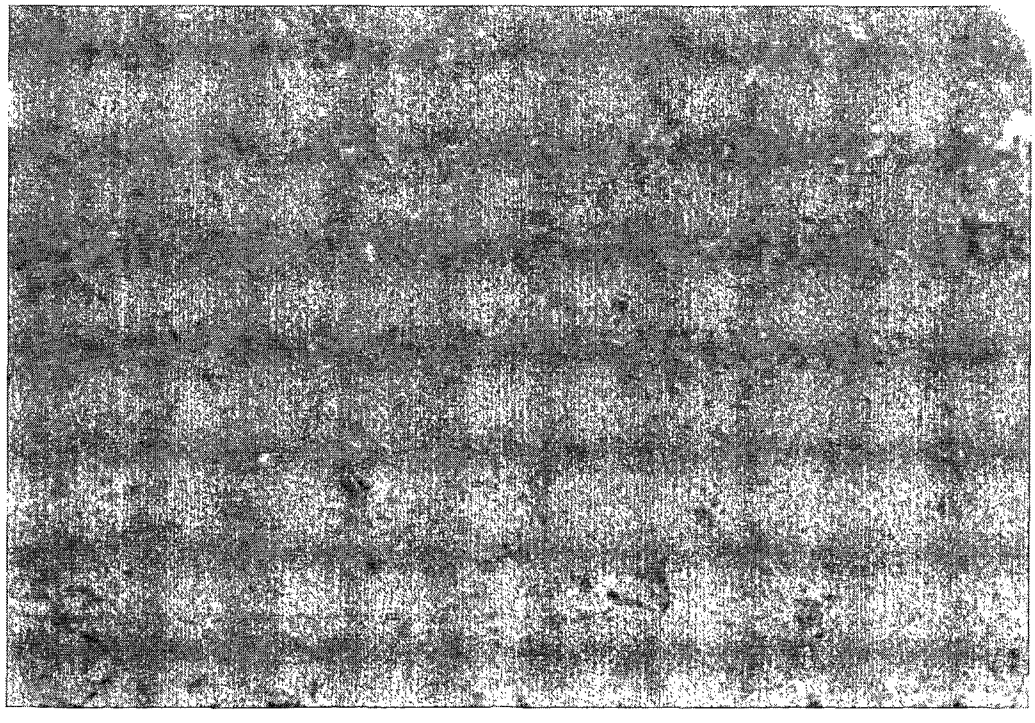
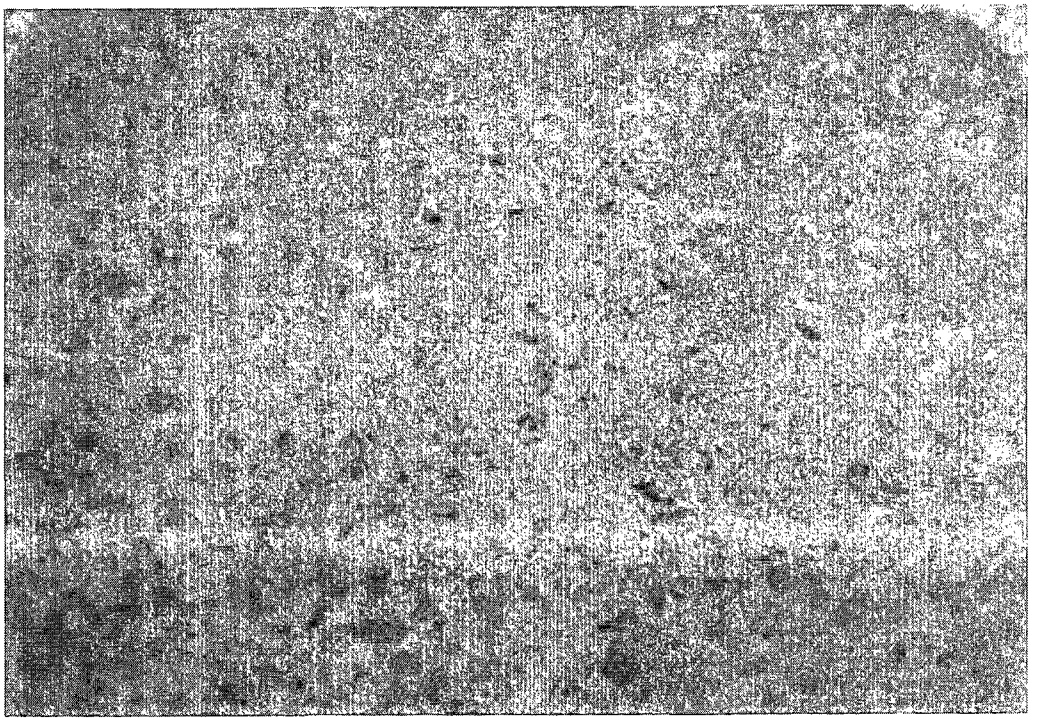
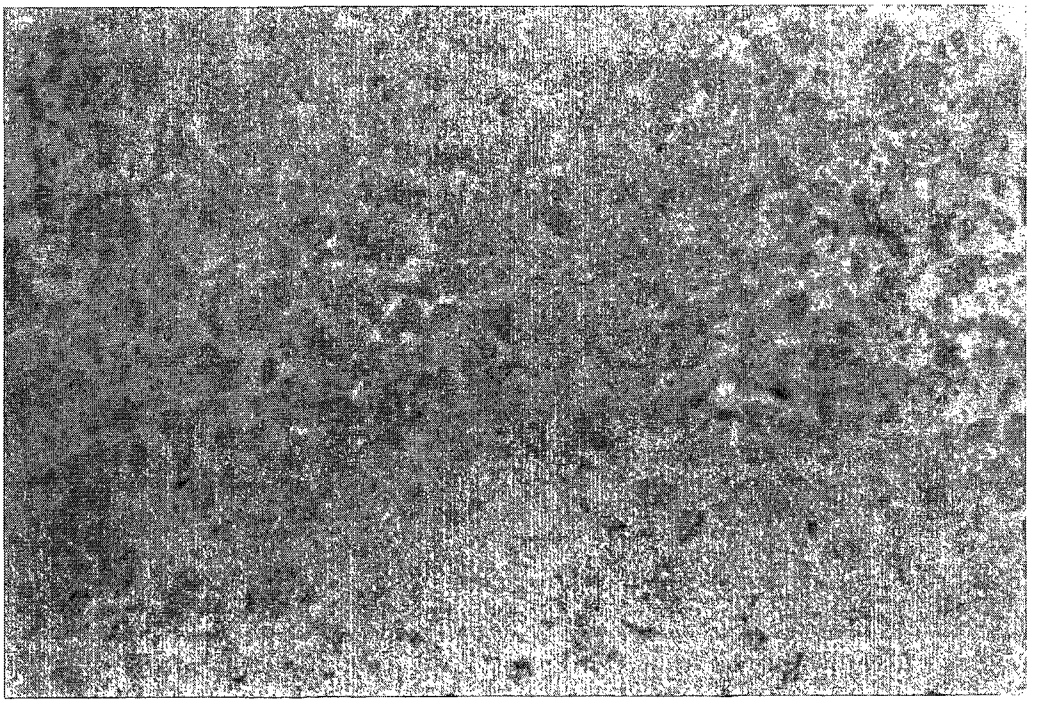


2nd sample

Figure C.2 --- Photos of the scaling specimens made of slag concrete according to ASTM C 672.

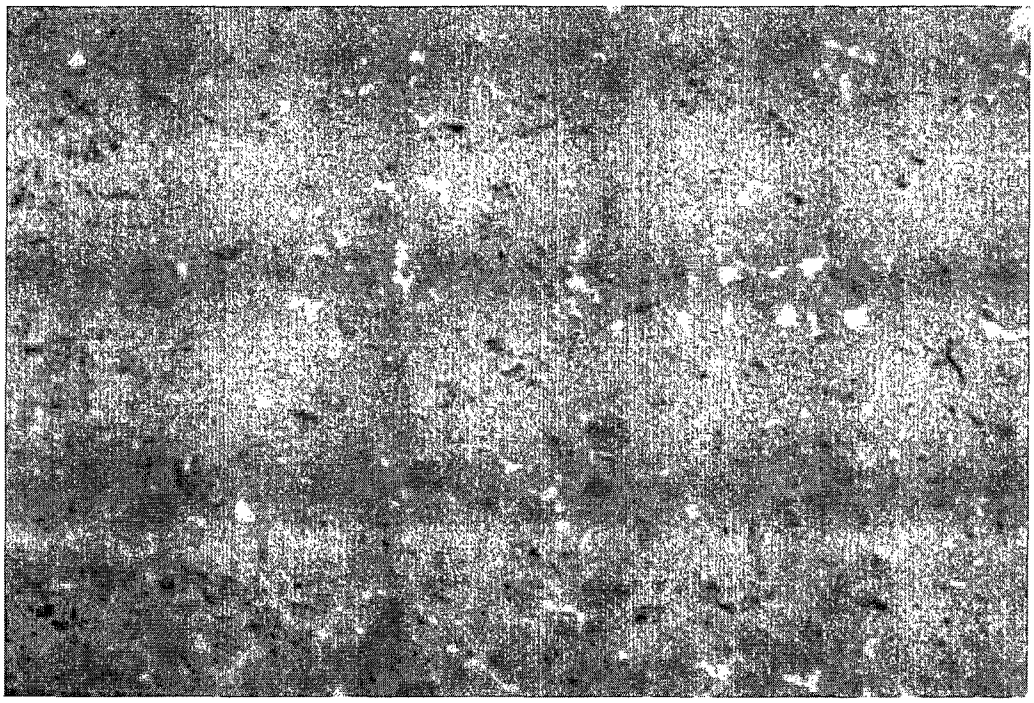


1st sample

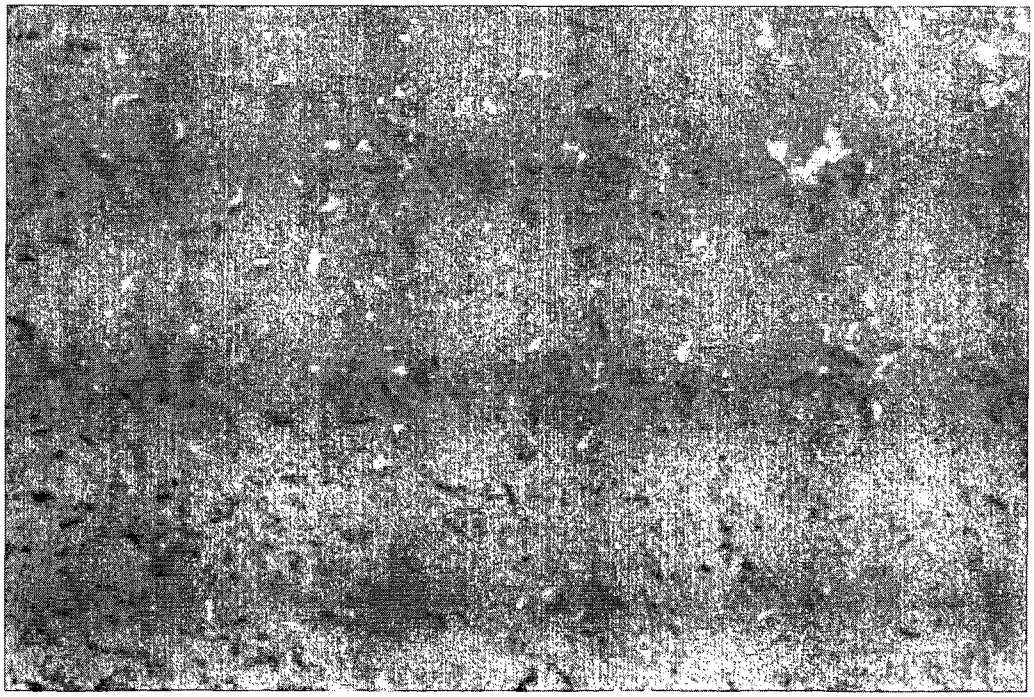


2nd sample

Figure C.3 --- Photos of the scaling specimens made of plain concrete according to ASTM C 672.

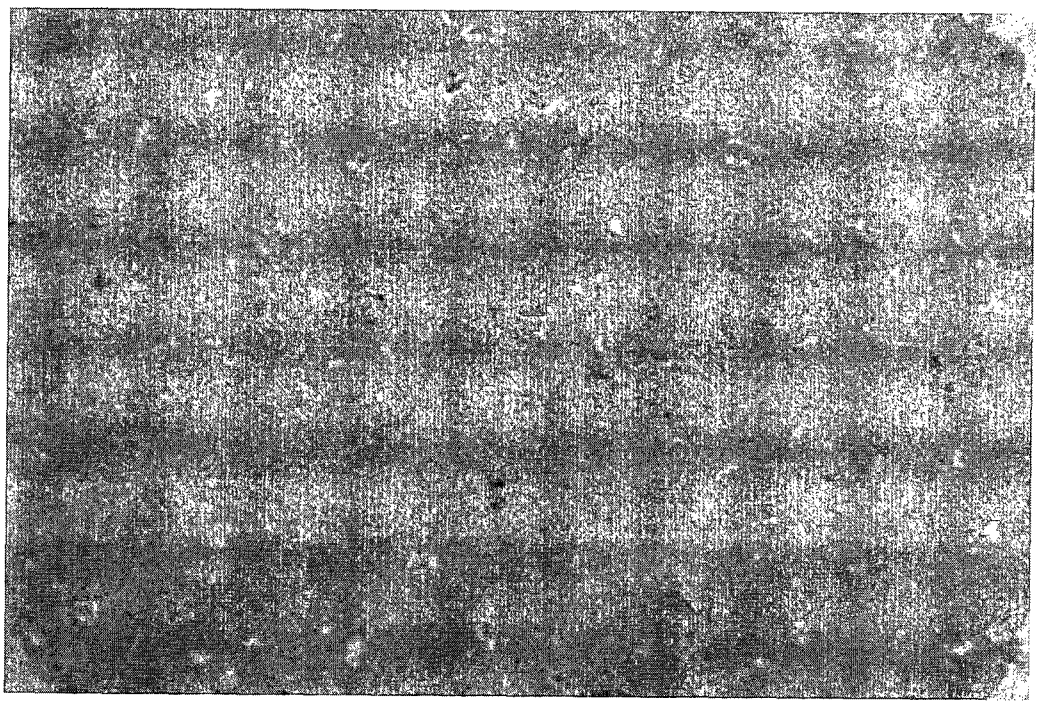


1st sample



2nd sample

Figure C.4 --- Photos of the scaling specimens made of fly ash concrete according to the proposed replacement method for ASTM C 672.



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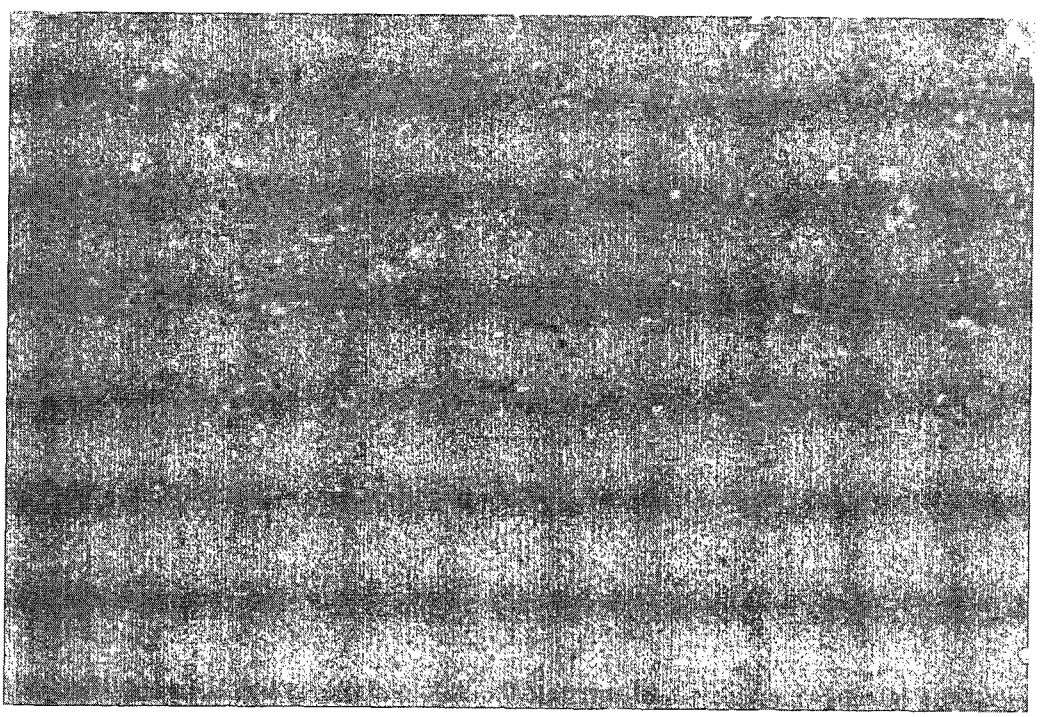
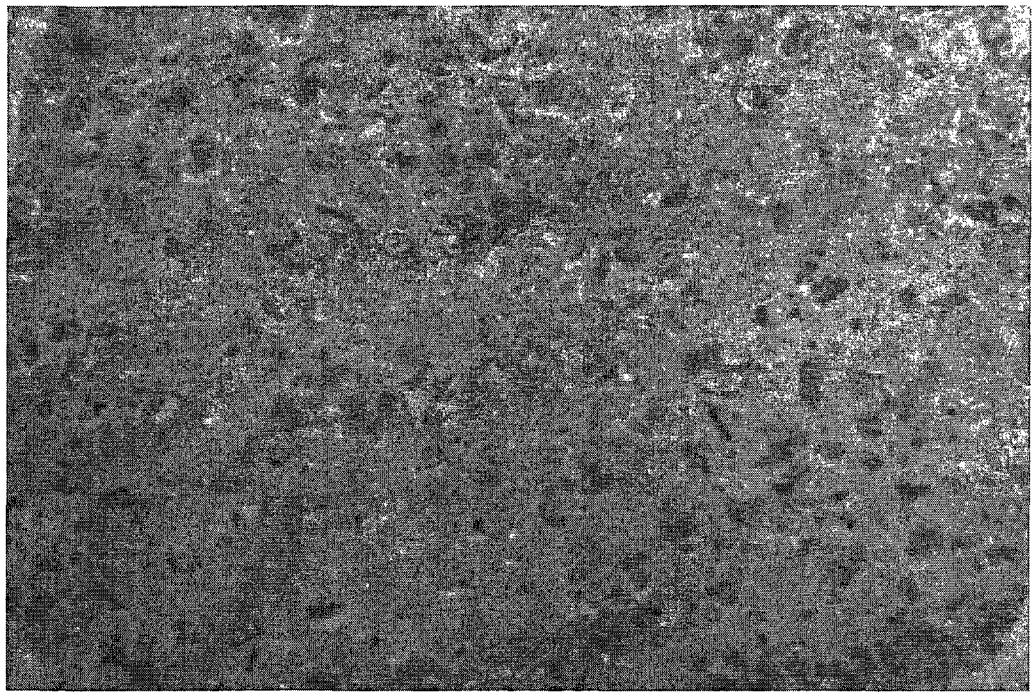
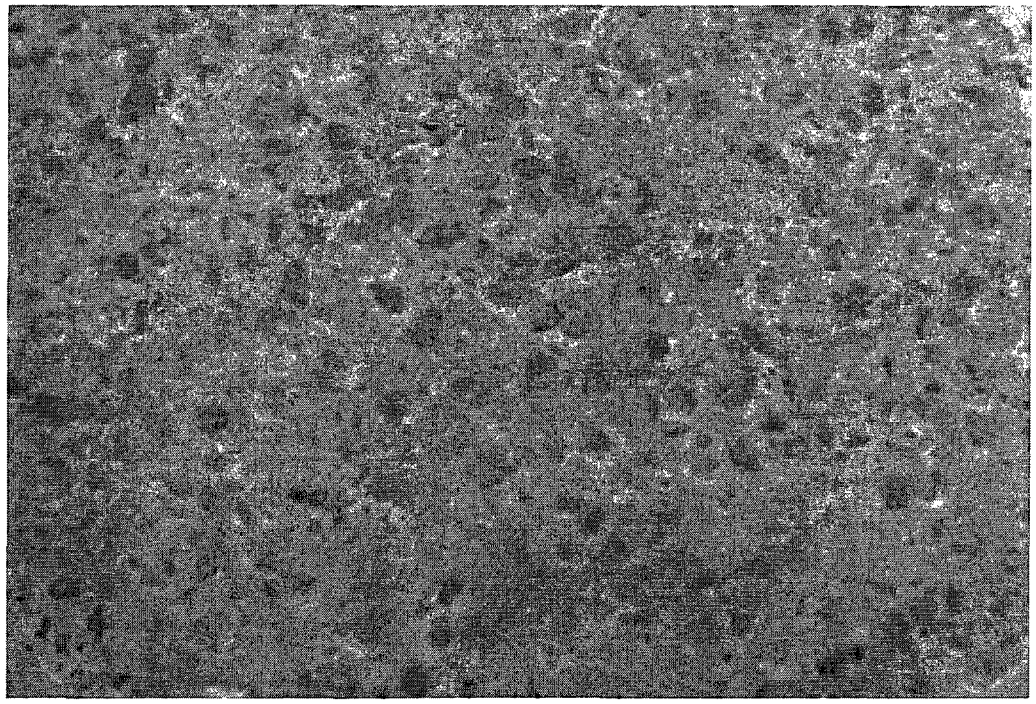


2nd sample

Figure C.5 --- Photos of the scaling specimens made of slag concrete according to the proposed replacement method for ASTM C 672.

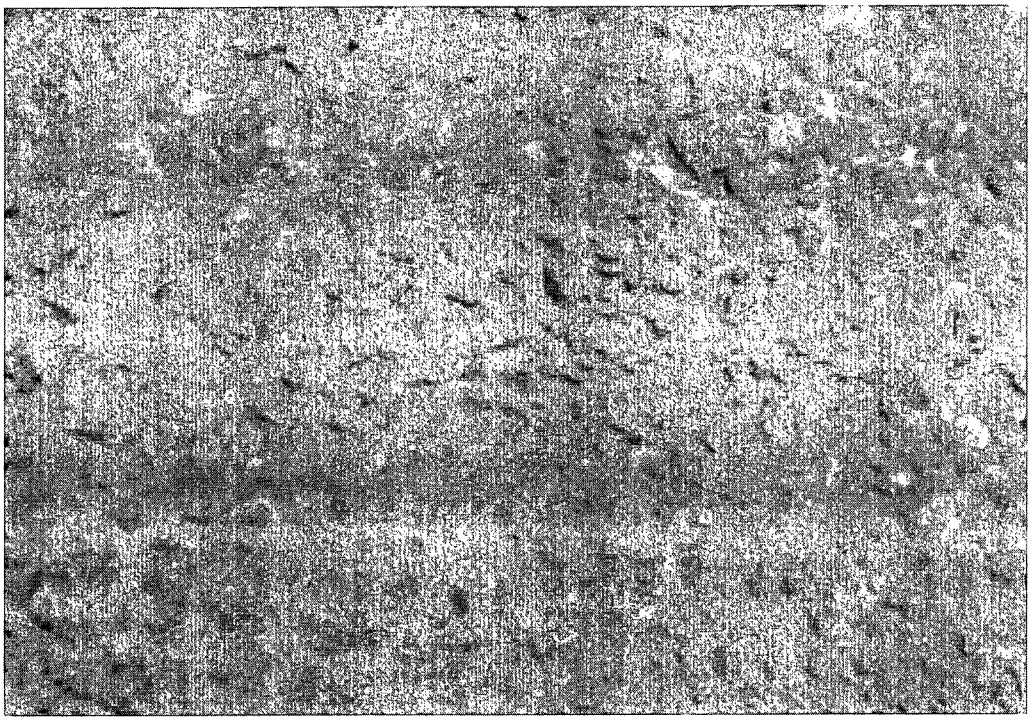


1st sample

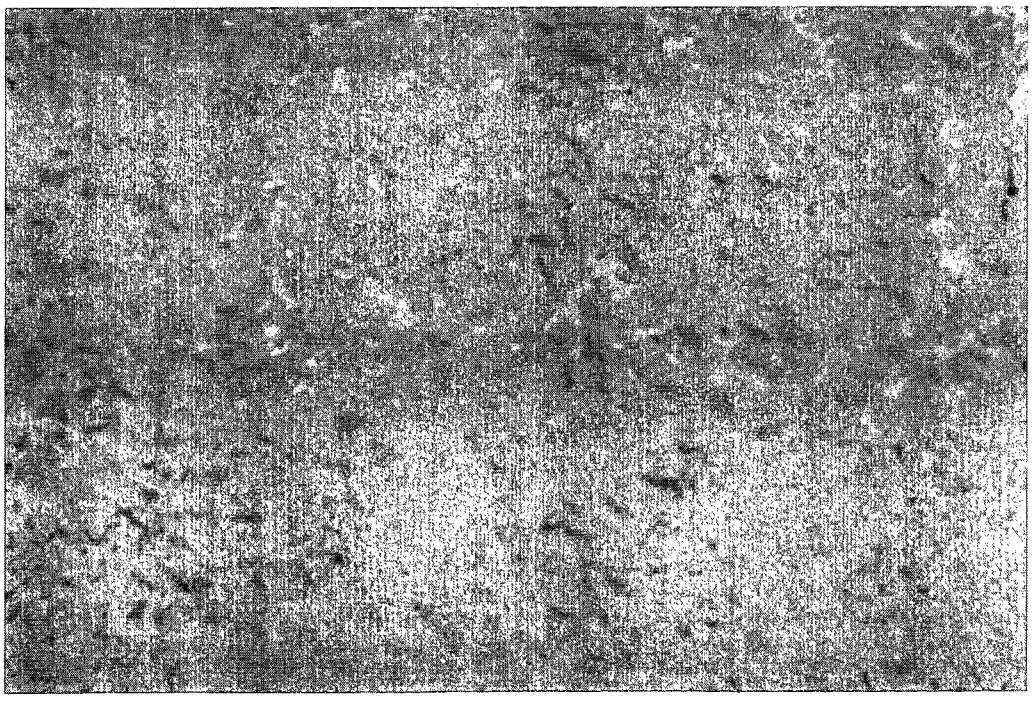


2nd sample

Figure C.6 --- Photos of the scaling specimens made of plain concrete according to the proposed replacement method for ASTM C 672.

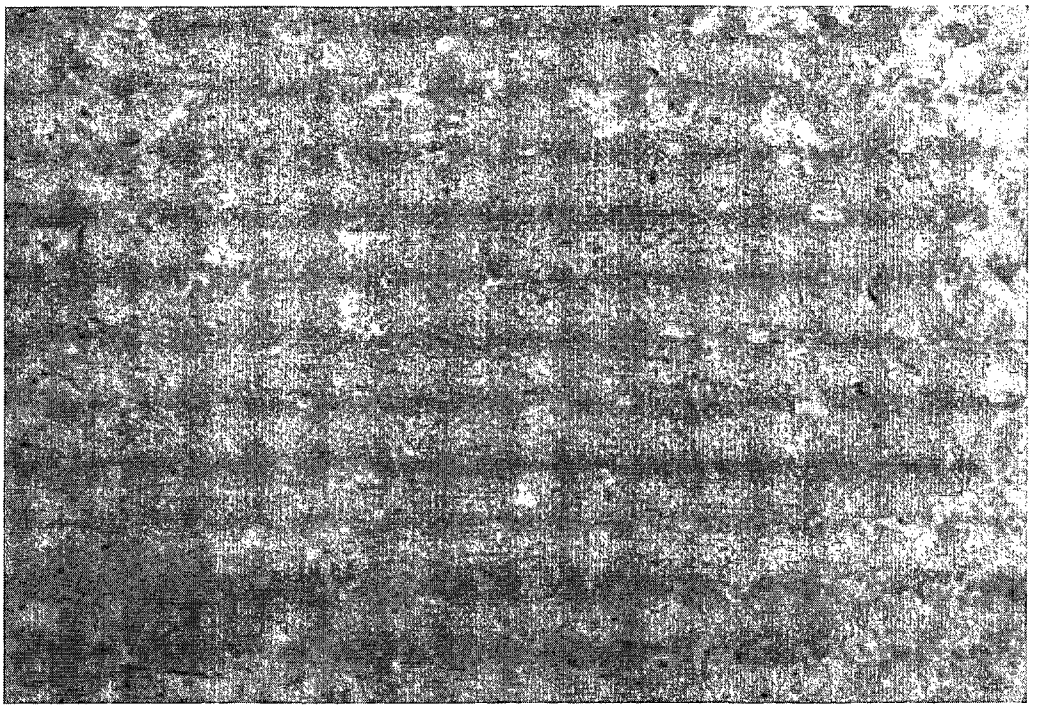


1st sample

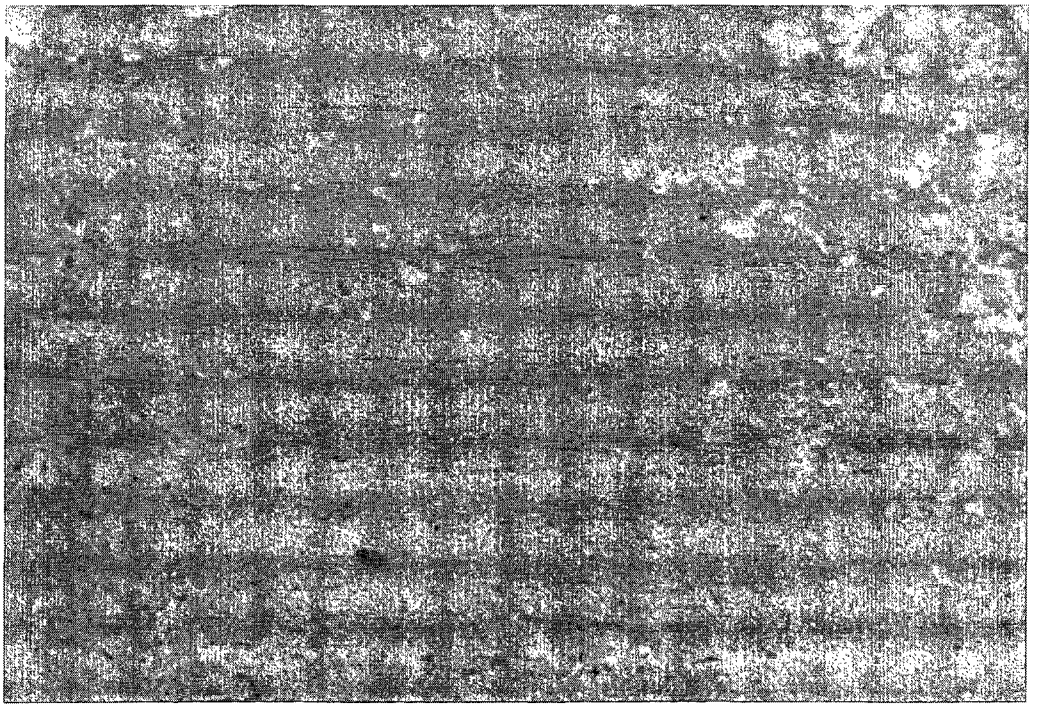


2nd sample

Figure C.7 — Photos of the scaling specimens made of fly ash concrete according to the method using the curing compound.

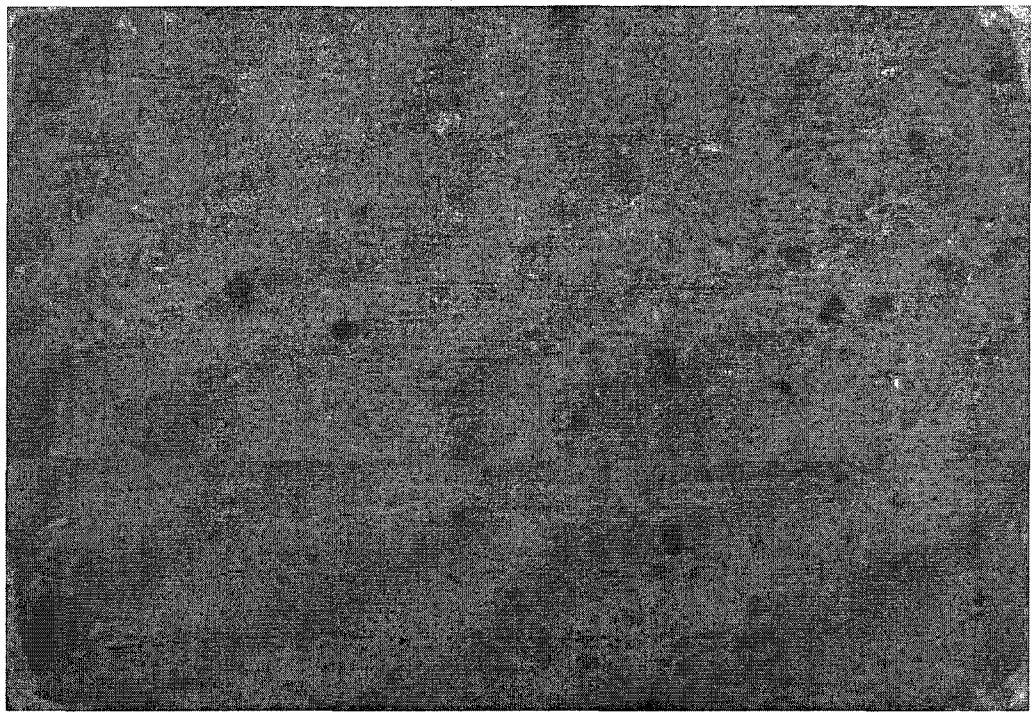


1st sample

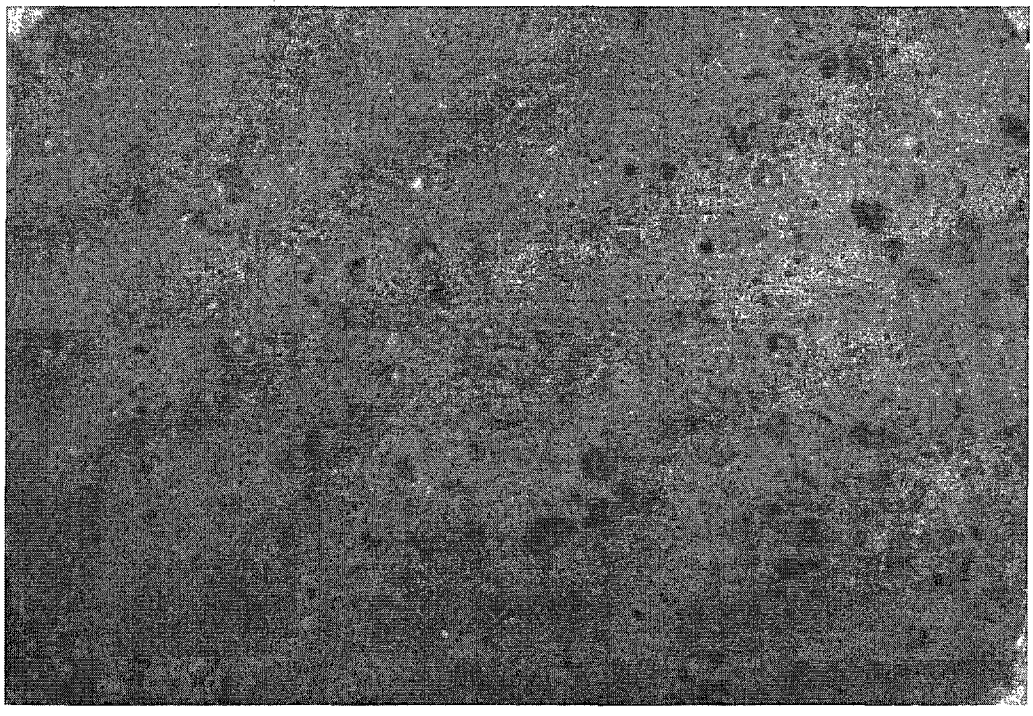


2nd sample

Figure C.8 --- Photos of the scaling specimens made of slag concrete according to the method using the curing compound.

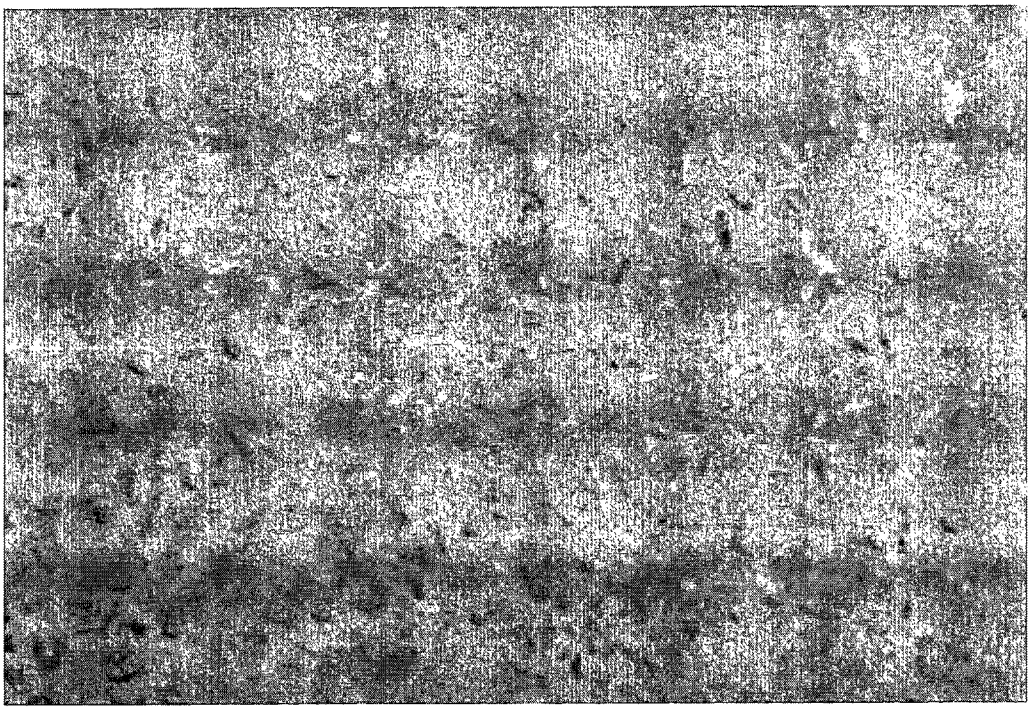


1st sample

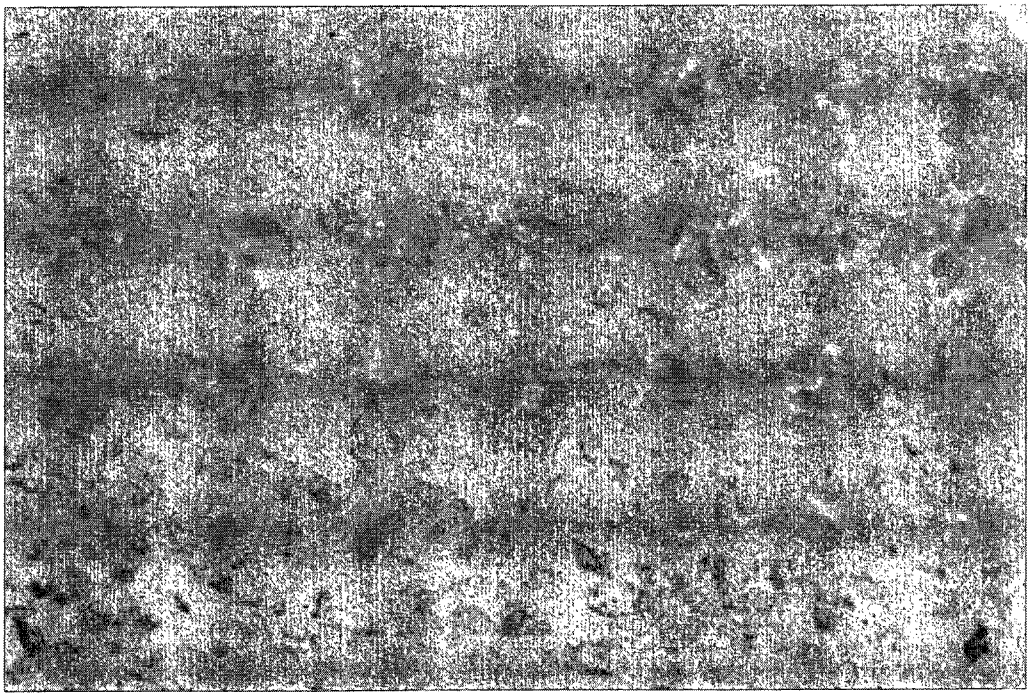


2nd sample

Figure C.9 --- Photos of the scaling specimens made of plain concrete according to the method using the curing compound.

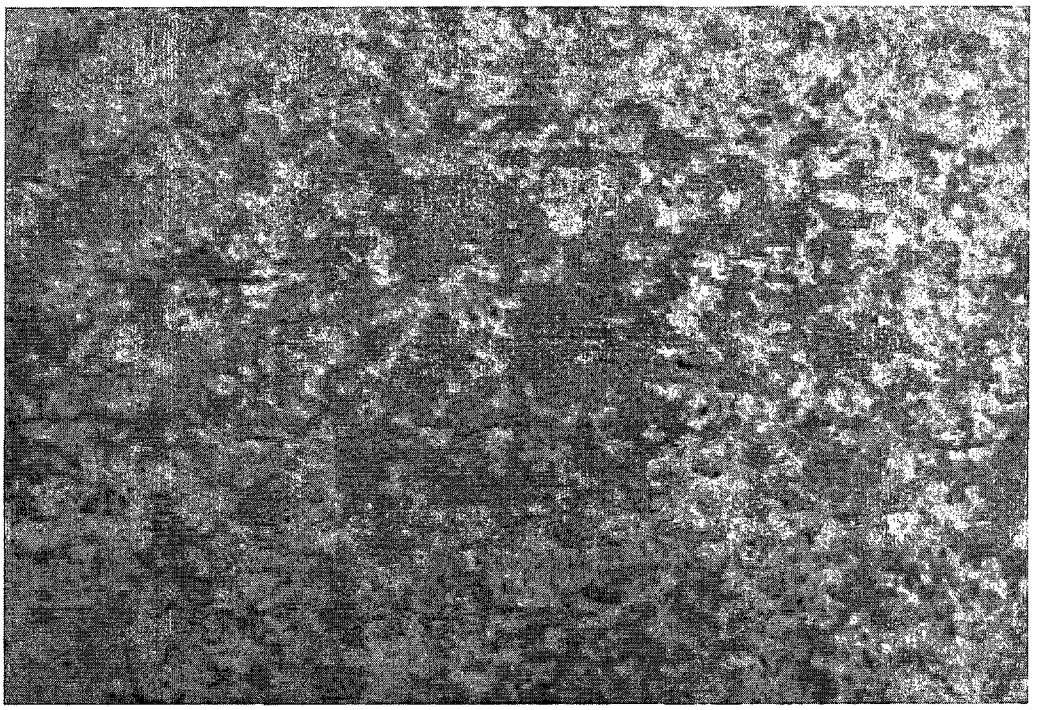


1st sample

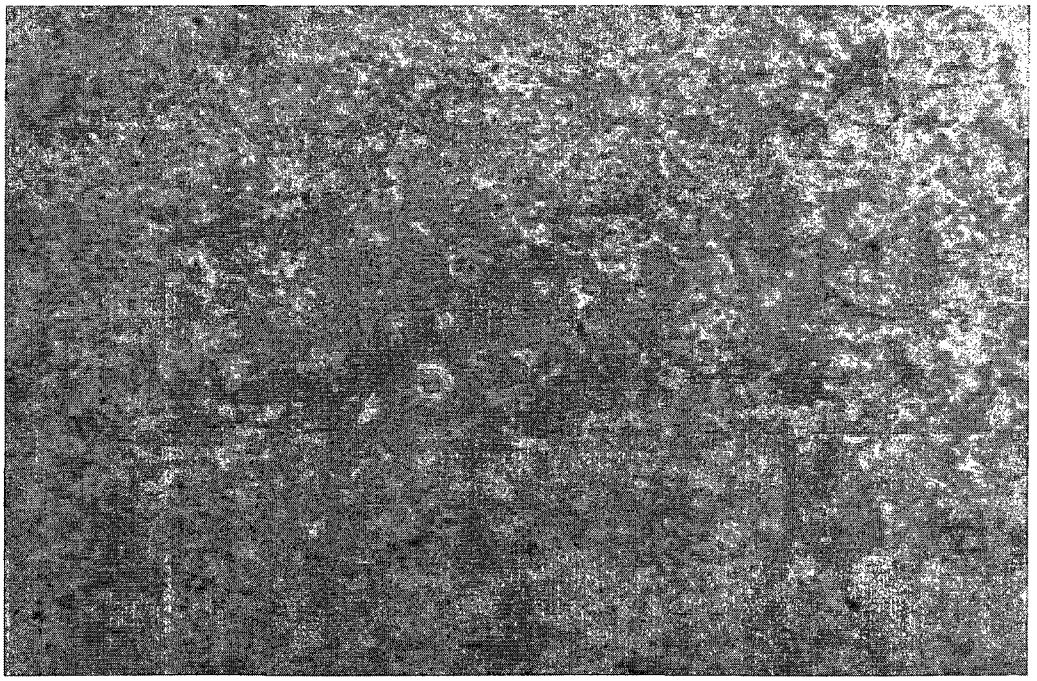


2nd sample

Figure C.10 — Photos of the scaling specimens made of fly ash according to the three-day curing method.

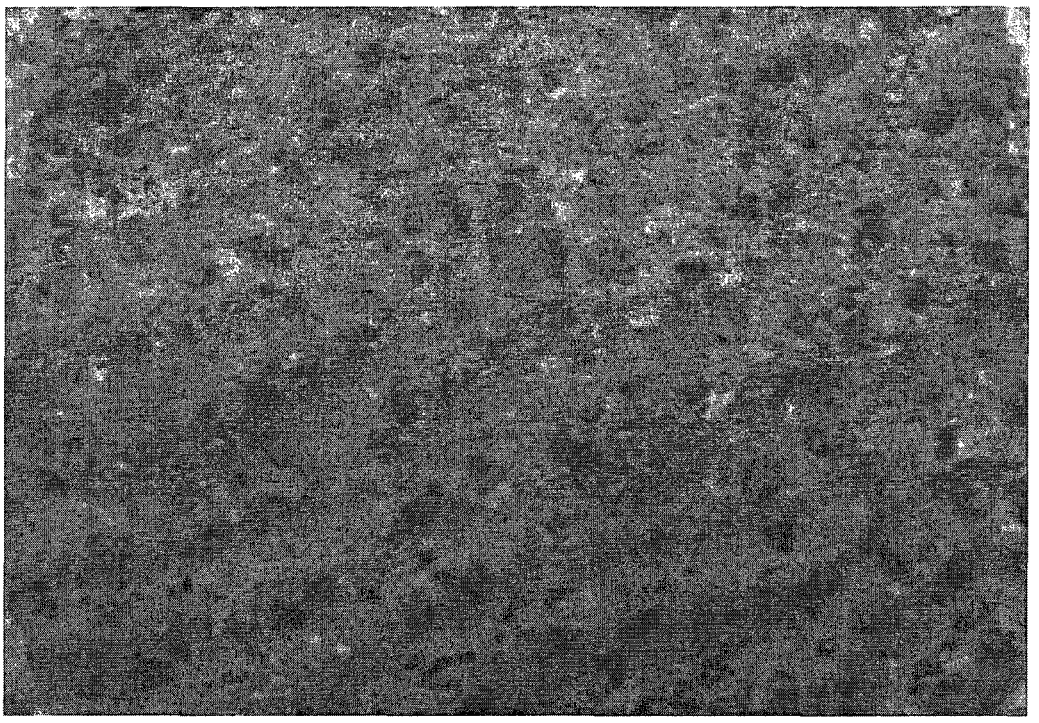


1st sample

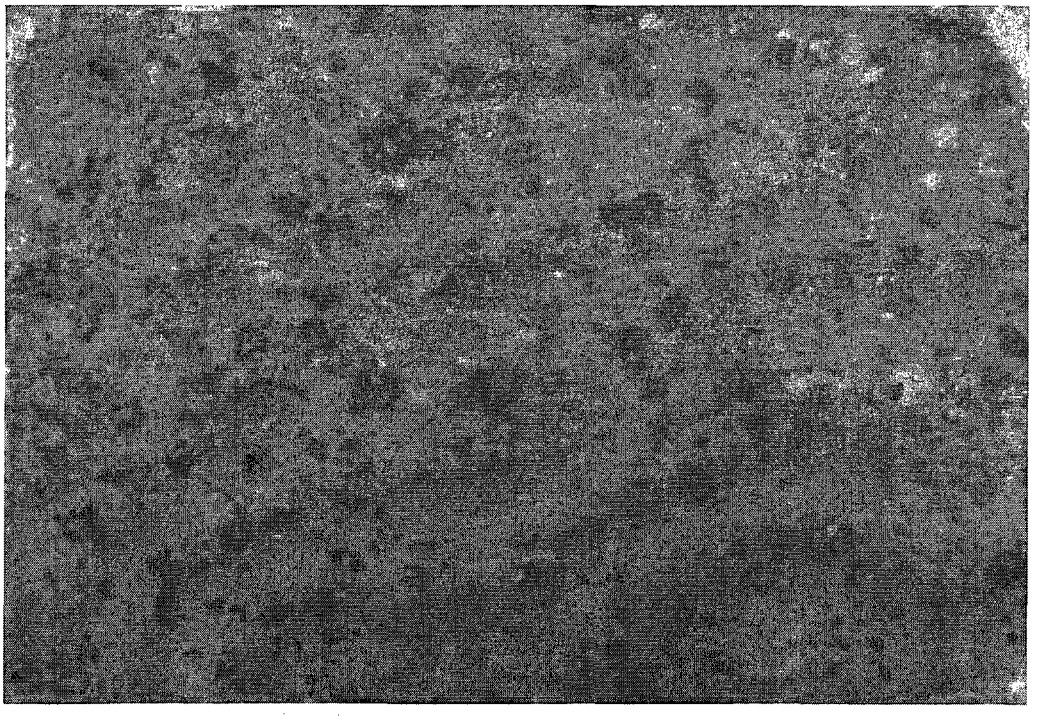


2nd sample

Figure C.11 --- Photos of the scaling specimens made of slag according to the three-day curing method.



1st sample



2nd sample

Figure C.12 --- Photos of the scaling specimens made of plain according to the three-day curing method.