

Review Article

Properties of Cementitious Repair Materials for Concrete Pavement

Shih Horng Yeo ¹, Kim Hung Mo ¹, Md. Akter Hosen ² and Hilmi Bin Mahmud¹

¹Department of Civil Engineering, Universiti Malaya, Kuala Lumpur 50603, Malaysia

²Department of Civil and Environmental Engineering, College of Engineering, Dhofar University, Salalah, Oman

Correspondence should be addressed to Kim Hung Mo; khmo@um.edu.my

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This review aims to provide an insight into concrete pavement repair materials by gathering the relevant research outcomes and specifications. It provides a systematic literature review that focuses on pavement repair and types of cementitious repair material. In general, conventional cementitious repair materials can be divided into 4 categories: ultra-rapid, rapid, accelerated, and normal strength gain. The mechanical and durability properties of conventional cementitious repair materials, i.e., calcium sulphoaluminate-based, calcium aluminate-based, polymer-modified concrete, and ordinary Portland cement-based repair materials are highlighted. The paper also discusses the newly developed ultra-high performance concrete and alkali-activated materials as potential repair materials for concrete pavement due to the good mechanical and durability performance. As a final point, this review may help readers to understand and determine the appropriate type of repair material when distress occurs in concrete pavement.

1. Introduction

According to Blanchard's American Highway Engineers' Handbook of 1919, the history of concrete pavement began in 1879 in Scotland [1]. Concrete pavement is commonly used in highways, airports, and bridge decks due to their ability to carry heavy loads and long-lasting performance. There are 7 types of concrete pavement widely constructed around the world, which include (1) jointed plain concrete pavement; (2) jointed reinforced concrete pavement; (3) continuously reinforced concrete pavement; (4) prestressed concrete pavement, (5) precast concrete pavement; (6) roller compacted concrete (RCC), pavement; and (7) pervious concrete pavement [2].

Concrete pavement is designed to have a longer life span with a lower frequency of future maintenance. A well-maintained concrete pavement can last for more than 40 years [3]. It was reported that the first American concrete pavement built in 1891 remains in service [4]. Apart from being more durable, concrete pavement is preferred in

certain situations such as laying directly over poor soils due to the uniform load distribution characteristic of the pavement [5, 6]. As a result, most of the airports rely on concrete pavements to construct the taxiway, runway, and apron, especially where heavy aircraft is involved.

Today, concrete pavement is widely constructed around the world. In 2018, the United States (US) Federal Highway Administration recorded around 90,030 km of rural and urban concrete pavement [7], while the concrete pavement length in Japan is 55,218 km [8]. Germany has approximately 25% of its major roads made of concrete, while in Belgium around 17% of its overall road (134,000 km) is concrete pavement. In Austria, two-thirds of high-volume motorways of about 4,000 km are concrete pavements [9]. France has been using concrete pavements for a long time, and it has 600 lane km of continuously reinforced concrete pavement [10]. In South-East Asia, Thailand has constructed more than 5,624 km of concrete road [11].

The growth of the concrete pavement network has led to the increment of concrete repair works. To ensure the

success of the repair jobs, material selection is crucial. The considerations include (1) material's strength (compressive strength, flexural strength, fatigue resistance, etc.); (2) volumetric stability (shrinkage and coefficient of thermal expansion); and (3) the ability to withstand environmental load (i.e. freeze-thaw attack, sulphate attack, de-icer chemical attack, etc.) [12]. Pavement repair materials made from magnesium phosphate, calcium aluminate cement, polymer-modified cementitious material, and high early strength cement can gain very high strength at an early age, therefore pavement can be put into service within hours (e.g., 3 hours) [13].

Newly developed materials such as ultra-high performance concrete (UHPC) and alkali-activated materials (AAM) have attracted much interest in the pavement repair industry. UHPC possesses superior mechanical properties such as compressive strength greater than 150 MPa and a post-cracking tensile strength greater than 5 MPa [14]. It is an effective repair material for bridge deck that is deteriorated by traffic load and freeze-thaw cycle [15]. AAM has gained popularity in the industry due to the reduction of carbon footprint. Unlike conventional AAM, recently developed AAM can gain strength without elevated temperature [16] and compressive strength as high as 145 MPa [17]. AAM is considered durable; the chloride ingress is similar to concrete with a low water-to-cement ratio (w/c) and it is more superior to concrete in terms of chemical attack [18]. Hence, the potential usage of AAM as a pavement repair material is excellent.

Although concrete repair material technology has grown substantially to date, limited reviews have been done about the cementitious repair materials for concrete pavement. The main objective of this review is to provide an insight into the types of cementitious pavement repair materials used for repair in the industry, mostly for partial- and full-depth repairs.

2. Pavement Distress

Distresses in the pavement are developed gradually over time due to cumulative fatigue load from traffic and environmental effects [19]. When the pavement is unable to perform, it has to undergo a repair process. The term repair in this paper refers to the work to restore a damaged pavement to its original or as-built condition in terms of strength and/or capacity.

Cement Concrete and Aggregates Australia categorises pavement distresses into 3 types, which are structural distress, durability distress, and functional distress [13], whilst NCHRP Report 747 groups distress into functional and structural distresses; durability issue is considered as structural distress [20]. Structural distress is related to cracking, joint deterioration, and punch-out. Once these occur, the traffic carrying capacity of the pavement is affected. Functional distress, which includes surface defects, roughness, and noise, will affect the ride quality and safety of the pavement.

The typical repair techniques for damaged concrete pavement involve full depth repair (FDR), partial depth

repair (PDR), and overlay [21]. As determined by the degree of damage, most of the functional distress and surface damage require PDR, overlay, and surface grinding. In the case of structural and durability distress, the recommended repair work is FDR or overlay. PDR is for rehabilitating shallow and localised surface defects, whereas FDR is conducted within the full depth of the pavement. This review focuses on the technology in cementitious material repair for PDR and FDR of concrete pavement.

3. Repair Material Requirements

Normally, a repair material is expected to last for 5–15 years [22]. The material must be compatible with the existing substrate to ensure that the material can function until the end of service life. Compatibility can be defined as a balance of physical, chemical, and electrochemical properties, as well as dimensions between repair material and the existing substrate. This is to ensure that the repair can withstand all the stresses induced by volume changes, chemical, and electrochemical effects without distress and deterioration over a designated period [23]. The general requirements of repair material to prevent property mismatch in the repair of structural concrete is shown in Table 1.

3.1. Strength. The mechanical properties of pavement repair materials are normally evaluated through their compressive strength or flexural strength. During repair, it is important to ensure that the repair material is not loaded prematurely; else it can have a detrimental effect on the long-term performance. Therefore, the repair material for pavement must meet certain strength criteria before opening to traffic.

3.1.1. PDR. For PDR, the material strength required to carry traffic is around 11 MPa–13 MPa [27]. The load-carrying mechanism of PDR is different from FDR where PDR only carries compressive stress and does not carry flexural stress or tensile stress. However, certain road authority requires the materials to carry flexural stress. The strength requirements for PDR are shown in Table 2.

3.1.2. FDR. The repair material for FDR is required to carry both compression and tension stresses. Hence, the strength required for open-to-traffic is much higher than that for PDR. In the US, the minimum opening strength varies with repair size, typically ranging from 13.8 MPa to 20.7 MPa (Table 3). In contrast, the United Kingdom highway authority has regulated the open-to-traffic strength to be 25 MPa regardless of the size [29]. Recent literature by Louisiana Transportation Research Center suggests that the strength can be reduced from 20.7 MPa to 13.8 MPa [30].

3.2. Volumetric Stability. Normally shrinkage occurs at different ages in the life of the material; concrete starts to shrink when the hydration process begins, and it can last for years. There are several types of shrinkage to be considered in cementitious repair materials, including autogenous

TABLE 1: General requirement for repair material [24–26].

Property	Relationship of repair mortar (R) to a concrete substrate (C)
Strength in compression, tension, and flexure	$R \geq C$
Modulus in compression, tension, and flexure	$R \sim C$
Poisson's ratio	$R \sim C$
Coefficient of thermal expansion	$R \sim C$
Adhesion in tension and shear	$R \geq C$
Curing and long-term shrinkage	$R \leq C$
Strain capacity	$R \geq C$
Creep coefficient (for repairs in compression)	$R \leq C$
Creep coefficient (for repairs in tension)	$R \geq C$
Fatigue performance	$R \geq C$
Chemical reactivity	$R \leq C$

TABLE 2: Strength requirements for PDR [27].

State	Compressive strength, MPa (psi)	Flexural strength, MPa (psi)
New York	10.5 (1527)	—
Kansas	12.4 (1800)	2.1 (300)
Missouri	11.0 (1600)	—
Michigan	12.4 (1800)	2.1 (300)
Minnesota	20.7 (3000)	3.1 (500)
Colorado	17.2(2500)	—
Nebraska	25.0 (3625)	—

TABLE 3: Minimum opening strength necessary for FDR [28].

Slab thickness, mm (in.)	Strength for opening to traffic, MPa (psi)			
	Repair length <3 m (10 ft)		Slab replacements	
	Compressive	3rd-point flexural	Compressive	3rd-point flexural
150 (6.0)	20.7 (3000)	3.4 (490)	24.8 (3600)	3.7 (540)
175 (7.0)	16.5 (2400)	2.6 (370)	18.6 (2700)	2.8 (410)
200 (8.0)	14.8 (2150)	2.3 (340)	14.8 (2150)	2.3 (340)
225 (9.0)	13.8 (2000)	1.9 (275)	13.8 (2000)	2.1 (300)
≥ 250 (≥ 10.0)	13.8 (2000)	1.7 (250)	13.8 (2000)	2.1 (300)

shrinkage, plastic shrinkage, and drying shrinkage. The magnitude of shrinkage is influenced by several factors such as binder content, water content, aggregates grading and types, as well as usage of admixtures [31–34].

3.2.1. Plastic Shrinkage. Plastic shrinkage occurs when water is lost by surface evaporation or suction by the underlying material. The severity of the contraction depends on the amount of water lost, which is influenced by ambient relative humidity, wind velocity, and temperature [35]. A few measures can be applied to reduce the occurrence of plastic shrinkage in the repair material, i.e., ensure that the evaporation rate does not exceed 1 kg/m^2 per hour and the subgrade must not be dry. The utilisation of a low w/c ratio repair mix or fibre reinforcement may help to reduce the occurrence of plastic shrinkage [12]. Concrete with a low w/c has a lower rate of capillary pressure and less tensile stress build-up when the concrete is fresh. The risk of plastic shrinkage is reduced if the water content is less than 150 kg/m^3 [36, 37]. The addition

of synthetic fibre is effective in controlling plastic shrinkage formation [38]. Synthetic fibres such as polyethylene terephthalate, glass, and polypropylene were added to concrete to reduce plastic shrinkage [39]. Polypropylene is the most efficient in restraining plastic shrinkage cracks, where the addition of 0.1% to 0.3% fibres is reported to reduce the cracks by 50%–99% compared to plain concrete [39, 40]. It is noted that all the repair materials should have proper curing.

3.2.2. Drying Shrinkage. Drying shrinkage occurs due to inadequate or improper curing. The volume of concrete changes when moisture is lost to the surrounding. When the material shrinks and is restrained, the tensile stress exerted can reach up to 6.90 MPa [21]. This amount of stress is higher than the tensile strength of a Grade 50 concrete (approximately 4 MPa to 5 MPa) [41]. High drying shrinkage of the repair materials can cause incompatibility failure [23]; the materials may de-bond if they shrink at a different rate, hence proper curing should be executed after the repair.

Normally, the maximum allowable shrinkage of repair material used in concrete pavement repair varies from $400 \mu\epsilon$ to $700 \mu\epsilon$ at 28 days [42–44]. However, NYSDOT only allows non-shrink repair material [45].

3.2.3. Autogenous Shrinkage. Similar to drying shrinkage, autogenous shrinkage appears within the paste fraction of the repair material. It is more prominent when the w/c ratio is less than 0.40 [46]. At low w/c, water is withdrawn from the capillary pores by the hitherto unhydrated cement, thus leading to shrinkage of the paste [35]. Normally, the mix design for repair works requires low water content to achieve high early strength, i.e., w/c ratio of less than 0.40; therefore the repair material may be susceptible to autogenous shrinkage cracks.

3.3. Durability. A repair material must be durable and able to maintain its integrity in the environment. It must be able to withstand common physical, chemical, and mechanical attacks experienced in concrete pavement [47]. Deterioration by physical attack includes the effects of high temperature or non-uniform thermal expansion (the differences in thermal expansion of aggregate and the hardened cement paste). Under repeated heating or cooling processes such as alternating freezing and thawing processes, concrete can eventually degrade. Deterioration due to chemical attacks can be in the form of internal and external attacks. The internal chemical attack is associated with the reactive aggregate problem, while the external chemical attack occurs mainly through the action of aggressive ions, such as chlorides, sulphates, or carbon dioxide, as well as many natural or man-made liquids and gases. Mechanical attacks are mostly due to accidental load and overloading.

3.3.1. Freeze-Thaw Requirement. When cementitious material is saturated and exposed to low temperatures, the water within the capillary pores freezes and expands. The produced stress is sufficient to rupture the surrounding material. The cumulative effect of freeze-thaw cycles may degrade the repair material [48].

To ensure that the repair material can withstand the freeze-thaw resistance, authorities have set the minimum durability factor to be 80% [45, 49] or 90% [42] after a minimum of 300 cycles in accordance with AASHTO T 161 Procedure A (ASTM C666). When tested with ASTM C1262, the percentage of weight loss in the repair material shall be less than 1.0% after 100 freeze/thaw cycles using a distilled water solution or less than 1.0% loss in weight after 50 freeze/thaw cycles when tested in accordance with ASTM C67 [50]. The percentage of weight loss in the repair material shall not be greater than 1.0% after 25 freeze/thaw cycles [45].

3.3.2. De-Icer Scaling/Deterioration Requirement. De-icer scaling/deterioration is typically characterised by scaling or crazing of the slab surface due to the repeated application of de-icing chemicals in a freeze-thaw environment. The

deterioration often appears within the paste fraction. As both osmotic pressures and thermal stress become more prominent, the freeze-thaw effects are magnified [51, 52]. De-icing salts or chemicals exert pressures within the concrete as well as reduce the ability to withstand them [26]. In the process of scaling, calcium hydroxide within the paste is dissolved and the pore system becomes coarser, hence leading to the formation of deleterious compounds [51].

To prevent de-icer scaling in the cementitious repair material, the minimum cement content of the concrete is fixed at 335 kg/m^3 and the maximum w/c ratio shall not exceed 0.45 [51]. When tested under ASTM C672, the scaling resistance of mortar material shall not be greater than 4.88 kg/m^2 , while concrete is limited to a maximum visual rating of 2 (slight to moderate scaling) [42]. For high-performance concrete, the limit for visual rating is 3 (moderate scaling) [45].

3.3.3. External Sulphate Attack Requirement. Sulphate attacks can result from external and internal sources. External sulphate attack agents may come from groundwater, soil, de-icing chemicals, etc. The sulphate ions penetrate the cementitious system to form ettringite and gypsum, which lead to expansion in the hardened cementitious matrix. Cracks are formed once the tensile strength of the paste is exceeded by the tensile stress created by crystallisation pressure [53]. Another form of damage may be due to the destabilisation of calcium silicate hydrate and calcium hydroxide, resulting in the formation of microcracks without significant expansion.

Prevention of external sulphate attack may be possible by reducing the tricalcium aluminate (C_3A) content in the cement to be less than 8% [54] or using pozzolanic materials to reduce the quantity of calcium hydroxide in the hydrated cement paste. A w/c ratio should be less than 0.45 to help mitigate external sulphate attacks [53]. When the water-soluble sulphate in dry soil is greater than 2.00% or in water is greater than 10,000 ppm, the maximum w/c ratio should be limited to 0.40 [55].

3.3.4. Internal Sulphate Attack Requirement. The internal sulphate attack is similar to the external sulphate attack, except that the source of the sulphate ions is from the internal. The sulphate ions may come from the cement, aggregate, mineral admixture, or the decomposition of primary ettringite when the concrete is cured at high temperatures. Ettringite is usually formed during cement hydration at an early stage. However, due to high concrete temperature, $>65^\circ\text{C}$ at an early age, the ettringite formation is suspended [56]. The constituents of ettringite are dispersed as monosulphate and calcium silicate hydrates. When the concrete is moist and the temperature becomes lower, these constituents form secondary ettringite in hardened concrete. Some repair materials may experience high temperatures due to high cement content, high ambient temperature, and heat generated from the thick slab. Therefore, to control internal sulphate attack, the equivalent alkali content of the repair material should be limited to 3 kg/m^3 , while C_3A in the clinker and SO_3 of the cement should be

less than 8% and 3%, respectively [57]. The replacement of ordinary Portland cement with supplementary cementitious material (more than 35% of blast furnace slag, 20% of fly ash, 20% of metakaolin, or 10% of silica fume) may decrease the risk of internal sulphate attack [54]. Proper temperature control shall be adopted to prevent the cementitious repair materials from exceeding the maximum temperature.

3.3.5. Coefficient of Thermal Expansion Requirement. The coefficient of thermal expansion (CoTE) is a property that quantifies the amount of a material that expands and contracts during temperature fluctuation [58]. Generally, the CoTE thermal expansion of repair material ranges from 7.7 to 15.3×10^{-6} per °C [59] and for normal concrete, it may range from 7.4 to 13×10^{-6} per °C [2]. The compatibility between the thermal properties of repair material and pavement is crucial, as incompatibility between materials can cause detrimental effects such as spalling. When a repair material is subjected to weathering, the temperature cycle causes expansion and contraction of the concrete. Once the tensile stress generated exceeds the tensile capacity of the repair material, cracks and debonding may appear [60]. Concrete having higher CoTE is less resistant to temperature changes than concrete with lower CoTE [51].

3.3.6. Chloride Penetration Requirement. Rapid chloride permeability test (RCPT) (ASTM C1202/AASHTO T 277) is a good indicator to reflect the material durability. The penetration of chloride is measured by the amount of charge passing through the material. A normal concrete (w/c ratio of 0.40–0.50) is expected to exhibit RCPT charge passed ranging between 2000 and 4000 coulombs. Cementitious material with a w/c ratio lower than 0.40 that has RCPT charge passed 1000–2000 coulombs is rated as low chloride permeability [61]. General permeability requirement ranges from 1500 to 2500 coulombs [12, 42, 43, 62]. There are certain cases where the permeability is kept below 1000 coulombs for better durability requirements [63]. However, RCPT results may be misleading if the repair material contains calcium nitrite [64].

4. Constituent of the Repair Materials

Nowadays, the allowable traffic closure time is normally kept minimal to prevent heavy traffic congestion. A lane can be reopened when it meets the strength requirement for traffic opening. Often the suitability of repair material for PDR depends on two major factors: (1) allowable traffic closure time and (2) material properties such as strength, shrinkage characteristic, CoTE, elastic modulus, and creep coefficient [23, 27, 65].

The minimum time for traffic opening relies on the speed of the repair material to achieve the strength requirement. When using a lower early opening strength patch mixture, the load carry capacity should be determined before being put into service [27]. The early strength repair material can be grouped into 4 types, which are ultra-rapid, rapid, accelerated, and normal strength development [66]. The

ultra-rapid repair material can attain sufficient compressive strength and be opened to traffic within 2–4 hours. These materials must be able to achieve 13.8 MPa compressive strength in 2 hours and 20.7 MPa compressive strength in 4 hours. For rapid repair materials, traffic can be opened for service within 6–8 hours. These materials must attain 20.7 MPa compressive strength in 8 hours and 27.6 MPa compressive strength in 3 days. The accelerated concrete repair material is designed to acquire the specified strength within 12 hours. From the perspective of long-term performance, normal concrete outperforms other repair materials. The drawback of normal concrete is that it requires several days to achieve sufficient strength. If no guidance is given, concrete should obtain a minimum of 24.8 MPa compressive strength before being opened to traffic. These non-rapid materials are good quality repair with highly desirable 28 days characteristics [43].

4.1. Ultra-Rapid Material. Generally, there are 4 types of cementitious ultra-rapid material, namely, magnesium phosphate-based, calcium aluminate-based, calcium sulphoaluminate-based, and polymer-modified concrete.

4.1.1. Magnesium Phosphate-Based. Magnesium phosphate cement (MPC) contains no ordinary Portland cement as a binder. It is chemically bonded phosphate ceramic through an acid-base reaction. Generally, the basic material is cationic metal and the acidic part is phosphate [67].

MPC is usually used in areas with very high traffic flow and requires early opening for traffic [68]. MPC repair material can achieve very high early strength (22.8 MPa) within a short period, e.g., 1 hour. The bonding test indicates that MPC exhibits good adhesion to parent concrete, where the bonding test shows that the 1-day and 7-days strengths are 7.17 MPa and 11.51 MPa, respectively [69]. The results comply with ASTM C928, the minimum bonding strength required for rapid setting materials is 7 MPa (1 day) and 10 MPa (7 days) [70].

As a pavement repair material, volumetric stability is essential. The MPC repair material has a similar CoTE as concrete (10×10^{-6} per °C) [41], which is 9.6×10^{-6} per °C [71]. It has lower shrinkage (less than 700 $\mu\epsilon$) compared to conventional ordinary Portland cement (OPC) mortar (approximate 900 $\mu\epsilon$) measured at 28 days [72]. Repair material with lower shrinkage and similar CoTE can have better compatibility. MPC repair material comes with high abrasion resistance, whereby Yang (2014) reported that the abrasion resistance is about double of the normal concrete. With reasonable air content, MPC repair material can perform under freeze-thaw conditions. Scaling of the repair only started after 36 freeze-thaw cycles, while it appears in normal concrete within 10 cycles. [71]. The RCPT result of magnesium phosphate concrete is around 3000 coulombs, while normal concrete is approximately 4000 coulombs. This indicates that magnesium phosphate concrete has a higher resistance to ionic transport [73].

However, the conventional MPC repair material is sensitive to moisture. When exposed to moist conditions, it

was observed that the mechanical properties of magnesium phosphate cement decrease with time [74]. Water can dilute the unreacted cement, thus increasing the porosity of the system [75]. To mitigate the effect of strength loss, ferroaluminates cement, calcium aluminate cement, or calcium sulphoaluminate cement were added to improve the water-resistant and stability of the magnesium phosphate system [76–78]. MPC is also not suitable to be used with limestone aggregate where the aggregate can accelerate the setting time, decrease the heat of evolution, and degrade the strength, which may be due to poor crystallinity and crystal morphologies of hydration products [79].

4.1.2. Calcium Aluminate-Based. Calcium aluminate-based pavement repair material is distinctive from other repair materials due to its early strength. When cured at 6 and 18°C, the 6 hours compressive strengths are 19 MPa and 24 MPa, respectively [80]. Calcium aluminate concrete (CAC) is a low shrinkage concrete, and it develops good bonding properties between the new and old substrate. The CAC may gain strength rapidly due to the formation of CAH_{10} and C_2AH_8 phases [81]. It is resistant to freeze-thaw cycles, as well as de-icing chemicals, besides having high abrasion resistance. It was reported that the CAC has approximately 10 times greater abrasion resistance than silica fume concrete. As a pavement repair material, CAC performs well under low temperatures. The limitation of calcium aluminate-based pavement repair material is strength loss when exposed to high temperatures. At elevated temperatures, the unstable CAH_{10} phase undergoes conversion to a porous C_3AH_6 phase. CAC concrete with 400 kg/m^3 binder and w/c ratio of 0.40 may suffer up to 47% strength loss after conversion [82]. To address this issue, the performance of the CAC should be evaluated with an accelerated conversion test to ensure the converted strength exceeds the required strength [83]. A recent guideline given by the Texas Department of Transport allows CAC to be used as a pavement repair material, provided the cement content is more than 400 kg/m^3 CAC and the maximum w/c ratio is limited to 0.35 [80]. Similarly, EN 14647 suggests that for structural purposes, the minimum cement content shall not be less than 400 kg/m^3 and the w/c ratio shall not be more than 0.40 [82]. The CAC pavement repair material must attain at least 20.7 MPa (cured at ambient temperature) at 3 hours and 27.6 MPa at 24 hours (cured adiabatically) [84].

4.1.3. Calcium Sulphoaluminate-Based. The calcium sulphoaluminate (CSA) repair material has gained popularity due to its rapid strength gain and volumetric stability [85]. CSA contains ye'elimite ($\text{C}_4\text{A}\overline{3}\overline{s}$) as a major constituent (30% to 70%). Unlike CAC, repair material made by CSA does not undergo a conversion process. It is an alternative binder to CAC. This high early strength repair material can achieve compressive strengths of 7 MPa to 28 MPa within 2 hours [86]. Other research has indicated that CSA concrete has a strength greater than 40 MPa at 6 hours [85]. In terms of volumetric stability, it was reported that the magnitude of shrinkage is half of that for high early strength Portland

cement [87]. Besides that, CSA-based concrete can perform at a temperature of under 5°C [88]. CSA-based concrete is a good anti-freeze material, where under freeze-thaw performance testing, CSA repair material was found to have 12% less weight loss compared to OPC concrete [89]. Also, CSA-based concrete exhibits good resistance to corrosion and chemical attacks such as sulphate, magnesium, and ammonium salts [81, 90]. Hargis et al. (2017) revealed that at the same w/c, the oxygen diffusion coefficient and water absorption of CSA concrete is 35% and 58%, respectively, of OPC concrete [91]. The electrical resistivity of CSA concrete is 21 times better than normal concrete [92]. Although there is concern about the risk of reinforcement corrosion due to the increment of chloride ions in pore solution when CSA concrete is exposed to chloride condition [93], Jen et al. (2017) has shown that by altering the calcium sulphate-to-ye'elimite ratio, the chloride-binding capability can be enhanced, which may help to resolve the issue [94]. According to Dam et al. (2005), the local authority of California has specified CSA pavement repair as its rapid repair material and requires CSA-based concrete to achieve at least 2.8 MPa flexural strength at 8 hours [51].

4.1.4. Polymer-Modified Concrete. Polymer-modified concrete (PMC) refers to the addition of latex (powder or liquid) to a cementitious-based material. When cured, the resulting concrete contains a continuous, interconnected matrix of latex polymer particles [13]. Common polymers such as acrylics, styrene acrylics (SA), vinyl acetate-ethylene (VAE) copolymers, styrene-butadiene rubber (SBR) co-polymer, vinyl ester of versatic acid (VeoVa), and epoxies are added into the cementitious material to improve the physical and durability properties [95].

PMC is considered as one of the cheapest high-performance pavement repair materials. It is often used in the area where traffic volume is very high and only allows short closure time, e.g., within 2 hours. The working temperature of this material is between 7 and 29°C. Normally, PMC is designed with a life span of 5 years [96].

PMC has high adhesion strength between concrete substrates [97]. The flexural and shear bond test results have revealed a bond strength increase of 32% and 120%, respectively, compared to normal concrete [98]. In terms of volumetric stability, Weng et al. reported that the drying shrinkage of polymer-modified cementitious repair material is below $200 \mu\epsilon$ [99], which is lower than most transport authority's requirements of 400–700 $\mu\epsilon$ [42–44]. The PMC repair material also exhibits good thermal compatibility, where the CoTE is around $10\text{--}20 \times 10^{-6}$ per °C, which is similar to normal concrete [100]. Apart from that, the high abrasion resistance and lower susceptibility to the freeze-thaw attack of PMC allow it to perform well under traffic exposure. As described by Mirza et al. (2002), the weight loss of PMC due to abrasion is only 28% of the silica fume concrete control sample. After 300 freeze-thaw cycles, the recorded weight loss of PMC is below 1%, while control samples recorded weight loss greater than 2% [101]. Lee et al. (2018) reported that the chloride ion penetration of PMC

repair material is less than 1500 coulombs at 28 days and less than 1000 coulombs when tested at 56 days [102]. Low ionic transportation of PMC indicates that it is a durable repair material.

4.2. Rapid Repair and Accelerated Repair Materials. Rapid and accelerated repair materials are made from high early strength Portland cement and normal cement. High early strength cement contains a high percentage of alite C_3S to give rapid hardening and setting properties [103]. In European standard, high early strength cement is categorised as “R,” while normal strength gain cement is denoted as “N” [104]; ASTM C150 classifies high early strength cement under Type III cement, while normal Portland cement as Type I cement [105]. The typical Blaine fineness of high early strength cement is $556 \text{ m}^2/\text{kg}$, while the Blaine fineness of OPC is $384 \text{ m}^2/\text{kg}$ [103]. Fast-track concrete normally consists of high cement content and low w/c ratio, which leads to early strength development. The summary of fast-track concrete is shown in Table 4. With the addition of accelerators in the concrete mix, the setting time can be shortened, and early strength can be achieved through the modification of cement hydration. The accelerator is commonly dosed at 1.25%–3.75% of cement content [106].

Generally, the accelerator can be in the form of soluble inorganic salts and soluble organic compounds. The most common accelerator is calcium chloride, which is inexpensive and readily available. However, it has a detrimental effect on steel reinforcement [111]. Non-chloride accelerators such as nitrates, thiocyanates, alkanolamines, formates, acetates, and alkaline hydroxides do not promote steel corrosion. Although there are various alternative accelerators, none of these outperforms the chloride-based accelerator. The chloride-based accelerator has the lowest cost-performance ratio compared to the non-chloride accelerator [111, 112].

The fast-track repair material comes with costs; it was reported that some high early strength repair materials may encounter poorer freeze-thaw performance. The amount of entrained air within the concrete is reduced due to the high fineness of cement and high-range water reducers. The performance can be further reduced if a calcium chloride accelerator was used [113]. The shrinkage of fast-track concrete is also higher due to its higher binder content. In contrast to freeze-thaw resistance, the wear resistance and resistance to external sulphate attack performed well due to the low w/c ratio and high binder content [108, 114].

4.3. Normal Strength Concrete. Normal strength concrete is made from cement, water, coarse and fine aggregates, admixtures, and sometimes with the addition of supplementary cementitious material. It can be easily designed based on guidelines provided by British Research Establishment or American Concrete Institute [115, 116]. It is known as concrete with strength less than 55 MPa [117]. Depending on the traffic and exposure, the required compressive strength of normal paving concrete ranges from 20 MPa to 55 MPa [49, 107, 118]. When the concrete is designed with normal

strength gain cement, the strength development is considered slow, unlike rapid gain strength repair material, which can achieve the required strength within several hours. The 1-day and 7-day strength is predicted to be approximately 34% and 78% of the 28-day strength, respectively [41]. Unlike accelerated repair material, normal strength concrete has lower binder content and higher aggregate content, hence it performs better in terms of shrinkage [21]. Table 5 shows the design mixes of normal strength concrete.

4.4. The Newly Developed Repair Material

4.4.1. Ultra-High Performance Concrete. In the year of 1994, Richard and Cheyrezy had successfully developed reactive powder concrete with compressive strength up to 800 MPa [122]. Since then, the race to develop ultra-high performance concrete (UHPC) has started. Although no clear definition of UHPC has been adopted, some researchers recognise UHPC as concrete to have a minimum compressive strength of 120 MPa with strain-hardening capability [123]. For structural UHPC, it is recommended that the concrete shall have at least 150 MPa compressive strength and with tensile strength greater than 6 MPa [14, 124]. The earlier version of UHPC is made of cement, silica fume, quartz sand, fibre, quartz powder, and superplasticizer [122, 125]. Later, “greener” UHPC has been developed, which incorporated fly ash, slag, and limestone powder as well as other eco-friendly binders [126–129]. A summary of the UHPC mix design is shown in Table 6.

The performance of UHPC has exceeded the basic requirements of conventional concrete pavement repair materials. It has very low porosity and improves fatigue behaviour, which is excellent for pavement repair. In short, UHPC has no significant failure after 10^6 load cycles [135]. The fatigue performance of UHPC is affected by the strength of the material, stress applied, fibre distribution, and testing specimen [136–138].

UHPC has very high early strength, which exceeds 50 MPa within 24 hours [139]. It is beneficial for the early opening-to-traffic. Surface damage due to abrasion is the least concern; it was reported that the weight loss of UHPC when subjected to abrasion resistance test is only 40% of that for ordinary concrete [140]. As a repair material, the bond between the original substrate and repair material can determine the effectiveness of the repair material. A field test has been carried out to determine the bonding between UHPC with the original substrate. Results indicate that adequate bonds are developed between the UHPC and the original substrate even without surface treatment (diamond grinding) to the substrate. However, the interface voids must be below 10% for better bonding to be developed and the substrate must be sound concrete. [141]. Figure 1 shows the interface between UHPC and the old substrate.

With high fibre content and low water-to-binder ratio, UHPC has lower drying shrinkage. However, the autogenous shrinkage of UHPC is higher than ordinary concrete. The autogenous shrinkage of UHPC can easily reach up to $1500 \mu\epsilon$. Hence, autogenous shrinkage dominates the total

TABLE 4: Typical ranges of constituent materials for high early strength concrete [51, 106–110].

Mix characteristic	4–6 hours concrete	6–8 hours concrete	12–16 hours concrete	20–24 hour concrete
Cement type	Normal or high early strength	Normal or high early strength or blended	Normal or high early strength or blended	Normal or high early strength or blended
Cement content	385–565 kg/m ³	385–565 kg/m ³	420–500	400–475 kg/m ³
W/c ratio	0.32–0.40	0.36–0.40	0.32–0.46	0.40–0.47
Accelerator	Yes	Yes	None to yes	None to yes

TABLE 5: Mix design for normal strength concrete.

CM	W	G	S	SP	AEA	W/cm	Compressive strength (MPa)	References
415	208	1049	729	—	0.0415	0.50	50	[119]
390	156	1297.8	566.2	1.95		0.40	50	
320	160	1286.4	633.6	1.92		0.50	40	[120]
280	168	1249.3	702.7	1.68		0.60	30	
400	200	930	873	4		0.50	40	[121]

CM: cement/silica fume/fly ash/ground granulated furnace slag; W: water; G: coarse aggregate; S: fine aggregate; SP: superplasticizer; AEA: air-entraining agent; w/cm: water/cementitious material.

shrinkage of the concrete. Instability in dimension caused by autogenous shrinkage leads to debonding of the repair material. Mitigation can be done by controlling the hydration rate, using shrinkage reducing admixture or expansive agent [142]. Providing more internal restraint within the concrete matrix may help to reduce autogenous shrinkage; stiff or high elastic modulus aggregates can restrain the shrinkage deformation [142]. It was reported that an increase in the total aggregate volume may decrease the autogenous shrinkage up to 38% [143]. Besides that, providing internal curing agents such as a superabsorbent polymer or moist porous aggregate may lead to lower autogenous shrinkage. Liu et al. (2019) has demonstrated that optimising UHPC with porous pumice can reduce the shrinkage from 700 $\mu\epsilon$ (E0) to less than 300 $\mu\epsilon$ (E1–E6), as shown in Figure 2 [144].

Due to the dense matrix, UHPC has lower porosity, which can be reflected by the chloride ion diffusion coefficient, D_{eff} . The D_{eff} of UHPC is in the range from 0.2×10^{-13} to 4.1×10^{-14} m²/s, while for normal grade 30 and grade 80 concretes, the D_{eff} is 1.1×10^{-12} m²/s and 6×10^{-13} m²/s, respectively [145]. Furthermore, it was noted that under 5–15 years of freeze-thaw cycles in marine condition, no deterioration was detected on UHPC. The depth of chloride penetration of UHPC is only one-third of high-performance concrete with water-to-binder of 0.33 and 8.5% silica fume [146]. After 1000 cycles of freeze-thaw in the salt solution, it was reported that the compressive strength decreases about 17%, while major strength loss of up to 41.6% only occurs after 1500 freeze-thaw cycles, indicating that the high resistance of UHPC in freeze-thaw condition [147]. In general, the freeze-thaw performance is influenced by the water-to-binder ratio, amount of supplementary cementitious material, and steel fibre. The optimum content of silica fume, fly ash, and steel fibre is 80–130 kg/m³, 60–100 kg/m³, and 25–50 kg/m³, respectively [148]. Apart from freeze-thaw resistance, UHPC shows low or no sign of scaling after the

test [149]. With its excellent performance, UHPC is a potential material for pavement repair material.

4.4.2. Alkali-Activated Materials. Sustainable development has emerged into a trend that leads to the development of “greener” construction material. Alkali-activated material (AAM) is an eco-efficient binder that is developed by activating an aluminosilicate precursor with an alkali medium. Common precursors are fly ash, slag, metakaolin, phosphorus slag, red mud, etc. The activators referred are sodium hydroxide, sodium silicate, sodium carbonate, etc. The usage of AAM can greatly reduce the carbon footprint up to 80% compared to OPC [150]. Table 7 shows the type of precursor and the resulting mechanical properties.

The early strength requirement is one of the crucial parameters for pavement repair. Conventional AAM requires high temperature for curing to acquire better early strength properties. A recent study shows that ternary blended AAM (slag, metakaolin, and fly ash) without high-temperature curing can achieve very high early strength, which exceeds 70 MPa within a day [152]. Thus, it is very useful for emergency repair. Electric arc furnace slag with fly ash as an additive in AAM could record more than 50 MPa within 7 days [159], and it may be used as a substitute for normal strength gain concrete repair material, which requires 28 days to achieve 50 MPa.

The performance of AAM is comparable with ordinary concrete or better when exposed to a severe environment [152, 160–162]. It was reported that the AAM outperforms ordinary concrete in abrasion resistance, and the depth of wear of AAM is approximately 40% of ordinary concrete [163]. Furthermore, volumetric stability is crucial for a repair material, as high shrinkage within the repair material may lead to debonding of the material with the original substrate. Although previous research indicates that the drying shrinkage of certain AAMs is 3–6 times higher than ordinary

TABLE 6: Mix design and performance of UHPC.

C	Materials (kg/m ³)											Mechanical properties (MPa)		References
	SF/PS/ NS	CC	G	QP	GGBS	FA	LP	SS/NSD	F	W	SP	Compressive strength	Flexural strength	
955	239	—	—	—	—	—	—	1051	191	153	13	170–230	25–60	[122]
1000	230	—	—	390	—	—	—	500	630	180	18	490–680	45–102	[130]
720	216	—	—	252	—	—	—	900	40	134	71	181.6	19.7	[131]
657	119	—	—	—	430	—	—	1050	—	—	166	100–180	13–25	[131]
582.1–600.0	24.3–25.0	—	—	—	—	259.9–267.9	—	1256.1–1294.6	—	173.2–147.8	43.3–44.6	—	—	—
596.1–614.9	24.8–25.6	—	—	—	266.1–274.5	—	—	1286.3–1326.8	—	177.4–151.5	44.4–45.8	100–120	10–20	[132]
592.6–611.2	24.7–25.5	—	—	—	—	264.6–272.9	—	1278.8–1318.8	—	176.4–150.6	44.1–45.5	—	—	—
824	107	—	0–70.57	—	—	—	—	1222	—	162	50–54	120–142	—	[133]
388–750	144	0–200	0–18.1	—	—	150–511	—	990	—	224	37	80–140	—	[134]

C: cement; SF: silica fume; PS: precipitated silica; NS: nano silica; CC: calcined clay; G: gypsum; QP: quartz powder; GGBS: ground granulated blast furnace slag; FA: fly ash; LP: limestone powder; SS: silica sand; NSD: natural sand; F: fibre; W: water; SP: superplasticizer

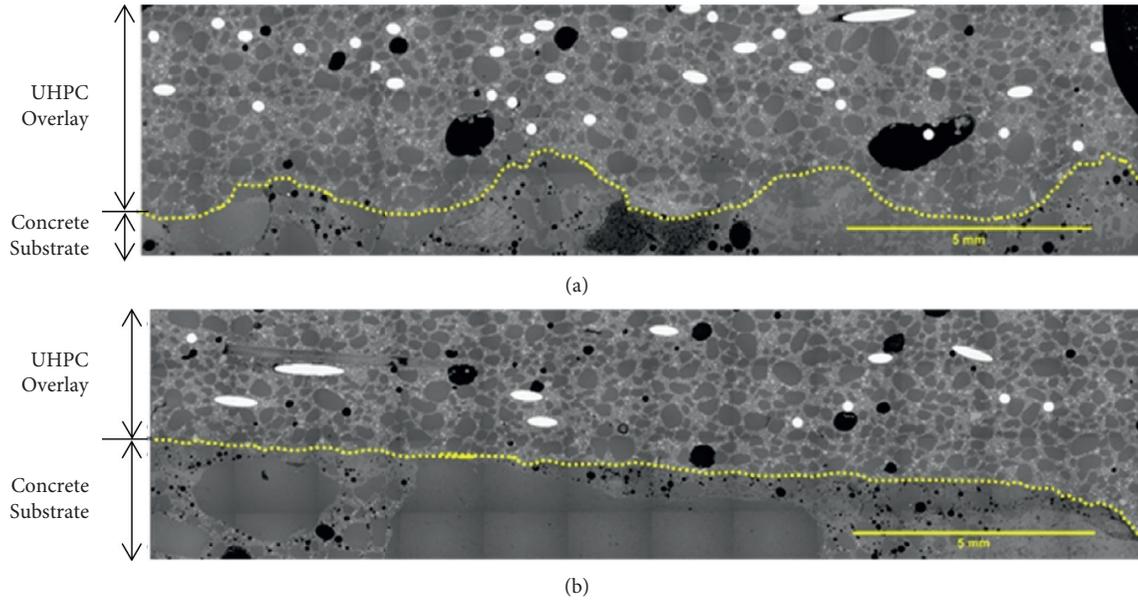


FIGURE 1: Interface between UHPC and substrate [141]. (a) Diamond ground surface. (b) No surface treatment.

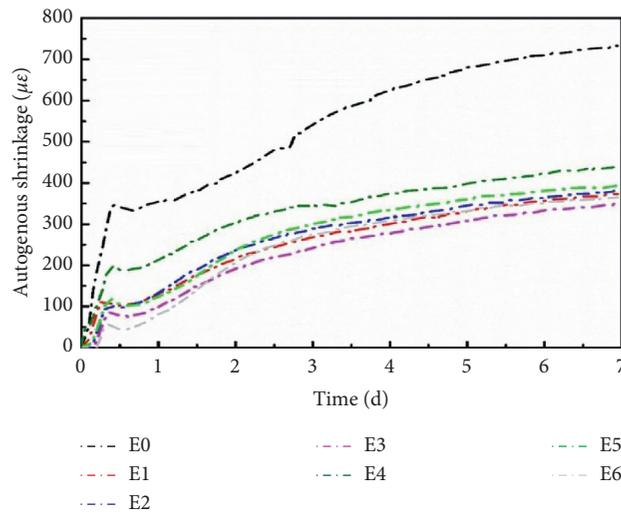


FIGURE 2: Autogenous shrinkage of UHPC with (E1–E6) and without porous aggregate (E0) [144].

TABLE 7: Type of AAM and mechanical performance.

Precursor	Activator	Liquid/binder	Curing condition	Compressive strength (MPa)	Flexural strength (MPa)	References
Fly ash/calcined carbonation lime residue	SS + SH	0.5	Room temperature	Approximately 35 MPa		[151]
Slag/metakaolin/fly ash	SS + SH	0.34	Room temperature	>80 MPa		[152]
Metakaolin/slag	SS + SH	0.7	50°C	63.6	10.6	[153]
High calcium fly ash	SS (anhydrous)	0.25	25°C	50		[154]
Ti-extracted residue/fly ash	SH	0.5	20°C	63.6	Approx. 6.5	[155]
Metakaolin/slag	SS + SH	0.5–1.1	Dry ambient curing/humidification/40 ± 5°C	41–60		[156]
GGBS/pumice/slag	SS + SH	0.5	20–25°C relative humidity: 40%	36.55–63.95	3.98–5.66	[157]
Metakaolin/fly ash/silica fume	PS + PH		25°C	80.7		[158]

SS: sodium silicate; SH: sodium hydroxide; PS: potassium silicate; PH: potassium hydroxide.

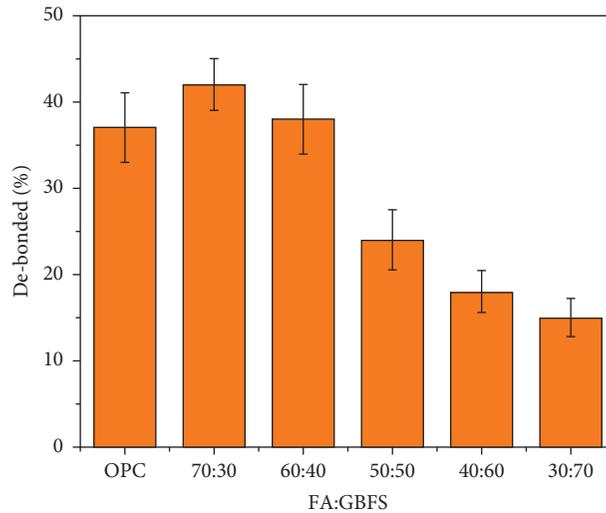


FIGURE 3: Percentage of de-bonded areas with different AAM composition [169].

concrete due to the activator [164, 165], mitigation can be done by incorporating shrinkage reducing admixture and gypsum [166, 167]. The incorporation of lightweight shale aggregate in AAM has proven to reduce the autogenous shrinkage by 50%–75% [168].

Recent research reported that the bonding between concrete and binary blended AAM (fly ash and slag) is encouraging. AAM with high slag content (Fly ash: Slag \geq 50:50) outperforms OPC repair material. Figure 3 indicates the percentage of de-bonded areas with different compositions of binary AAM [169].

Failure of repair material is often related to the ability to withstand the environmental load. Binary AAM with high content of slag performs better and retains more residual strength after the freeze-thaw cycle [169, 170]. Thus, the potential of AAM as a repair material for concrete pavement is promising.

5. Conclusions

This paper reviewed the different types of cementitious repair materials. This was done by gathering 171 different research articles, guidelines, and standards related to concrete pavement. The following are the major conclusions derived from this study:

Repair materials are expected to have a service life between 5 and 15 years, hence a repair material needs to be designed to be compatible with the original substrate. Compatibility of the repair material includes mechanical properties (strength, modulus, adhesion, and strain capacity), dimension stability (Poisson's ratio, thermal stability, and shrinkage), and long-term performance (creep, fatigue, and chemical reactivity).

A repair material must fulfill certain strength requirements before being opened to traffic. For PDR, the required compressive strength ranges from 11 MPa to 13 MPa. The required compressive strength for FDR is influenced by the repair size, and this ranges from 13.8 MPa to 20.7 MPa or flexural strength of approximately 4.5 MPa.

The volumetric stability of the repair material is crucial. For a repair material, it is normally designed with high binder content and low w/c. Hence, it is susceptible to plastic, drying, and autogenous shrinkage. Normally, the drying shrinkage is limited to 0.04%–0.07% at 28 days.

The minimum durability factor for freeze-thaw resistance is at least 80% in accordance with AASHTO T 161 Procedure A. If tested according to ASTM C1262 or ASTM C67, the percentage of weight loss in the repair material shall be less than 1.0% after the freeze-thaw cycles.

The scaling resistance of mortar material shall not be greater than 4.88 kg/m² when tested with ASTM C672. For normal concrete, the visual rating is limited to 2 (slight to moderate scaling), while it is limited to 3 (moderate scaling) for high-performance concrete.

To prevent external sulphate attack, the repair concrete must have a maximum w/c of 0.45. For internal sulphate attack, the equivalent alkali content within the repair material should be limited to 3 kg/m³, while C₃A in the clinker and SO₃ of the cement should be less than 8% and 3%, respectively. Suitable cement replacement can be adopted to reduce the risk of internal sulphate attack.

The CoTE of the repair material and original substrate (normal concrete) must be similar to prevent debonding of the material. A durable repair material can be reflected by the permeability which is normally limited between 1500 and 2500 coulombs when tested with a RCPT.

A repair material can be categorised by the speed of gain strength and can be grouped into ultra-rapid, rapid, accelerated, and normal strength repair material.

Depending on the area of usage, for emergency repairs such as an airfield, road junction, or high-volume freeway, the preferred material would be ultra-rapid material (2–4 hours traffic opening time), which can be made from magnesium phosphate, calcium aluminate, calcium sulphoaluminate, and polymer-modified cementitious binder.

In the event where traffic is required to open within 4–24 hours, rapid and accelerated repair material may be used. These materials are made of high early strength cement with

accelerator and designed with higher cement content. When traffic opening time is greater than 6 hours, blended cement with fly ash and slag may be incorporated into the concrete mix. The required binder content can be decreased with increased opening time. The high dosage of binder content and low water-binder ratio lead to an increase in the total shrinkage due to autogenous shrinkage and drying shrinkage.

For better long-term performance, concrete pavement repair with normal concrete is the best solution. Normal concrete is accompanied by lower binder content. Hence, it is more stable in terms of volumetric changes and can be more compatible with the existing pavement material. Also, normal concrete can be readily available and at a lower price.

Newly produced construction materials such as UHPC and AAM have high potential to be used as concrete pavement repair material. Besides the higher mechanical properties such as compressive and flexural strengths, they are more durable than normal concrete when exposed to a harsh environment as reflected by the better abrasion and freeze-thaw resistance. Both materials also have good bonding between the original substrate, which can ensure no debonding issue.

Data Availability

No data were used to support this study.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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