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Performance of soft-hard-soft (SHS) cement based composite subjected to blast loading with consideration of interface properties.

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1 that the SHS composite exhibited high resistance against blast loading, as per its
2 design.

3 **KEYWORDS:** numerical model; SHS multi-layer composite; high strength
4 concrete; blast load; dynamic increase factor; strain rate effect; interface

5 **1. INTRODUCTION**

6 Current runway pavement systems are designed for normal aircraft landing and
7 taking off, and may not be adequate to provide the required resistance to blast loads
8 arising from manmade/military bombing attacks and aircraft crashes. If the pavement is
9 damaged, repairs must be rapid to minimize service disruptions. It is even more
10 desirable that the extent of the damage be kept to a minimum, making rapid repair more
11 viable. Existing pavement materials such as concrete and asphalt are not able to provide
12 adequate resistance against blast loads. Due to their relatively brittle behaviour and
13 limited penetration resistance, conventional pavements are not protective and damage
14 caused by explosives might be too violent to be mitigated. As such, it is important to
15 propose a new pavement system to sustain blast load.

16 Some construction materials, such as high strength concrete (HSC) (Dancygier
17 and Yankelevsky 1996, Zhang *et al.* 2005), engineered cementitious composites (ECC)
18 (Li *et al.* 1994, Li and Maalej 1996), and geosynthetics (GST) (Koerner 1998), have
19 shown unique characteristics of high hardness, toughness and tensile strength. Such
20 materials have great potential to be incorporated into a new pavement material to
21 enhance the blast resistance of the pavement system. However, when considering the
22 strength and weakness of each material, it is difficult for one single material to fulfil all
23 of the requirements to resist blast load. After such consideration, a basic configuration
24 of a multi-layer composite system was proposed to fully utilize each individual

1 material's benefits under blast loading. This composite system consisted of three layers,
2 soft, hard, and another soft material layer. The "soft" material, asphalt concrete [AC],
3 was used as a sacrificial surface layer to absorb part of the dynamic energy. Thereby
4 greatly reducing the energy transmitted to the following layers. With the inclusion of
5 high strength GST as a reinforcement for AC, the tensile strength of the whole AC layer
6 was increased, which, in turn, reduced the extent of cracking and local failures within
7 the AC layer when subjected to dynamic loading. A "hard" material was used under the
8 AC layer, serving as the main body to sustain dynamic loads. HSC was proposed as a
9 good candidate due to its high compressive strength. Under dynamic loading, tensile
10 stress tends to develop at the rear face of the material due to the reflection of the
11 compressive stress propagating from the top face. However, it is well known that HSC
12 has a low tensile strength and is very brittle compared to conventional concrete.
13 Therefore, another "soft" and ductile material was deemed necessary at the base of the
14 "hard" HSC layer to absorb the reflected energy. ECC was proposed to take the soft
15 material role in this study, as it is classed as an ultra ductile fiber reinforced
16 cementitious composite due to its exceptional multiple cracking performance. This
17 composite is referred to as SHS composite hereafter. As such, it is necessary to fully
18 understand SHS composite's resistance against blast loads.

19 Full-scale field blast tests are time-consuming and costly. Therefore, reliable
20 numerical simulation could be considered the best alternative. Hence, this study aimed
21 to develop a 3D model to simulate the SHS composite using the explicit non-linear
22 finite element program, LS-DYNA. A field blast test was undertaken and used for
23 verification of the numerical model. For the numerical model, considering the important
24 role of the material model in modelling different layers, the suitability of the existing
25 material models in LS-DYNA (2007) was therefore performed. Laboratory experiments

1 were also conducted to validate and quantify relevant parameters. Strain rate effect was
2 considered and dynamic increase factor (DIF) of relevant materials were also developed
3 and incorporated into the model. Furthermore, the interface properties were investigated
4 and verified through experiments and the results incorporated into the 3D model.

5 **2. FINITE ELEMENT MODELLING**

6 **2.1 LS-DYNA**

7 The numerical study on the SHS composite subjected to blast loading was
8 performed using the LS-DYNA, an explicit finite element code dedicated to analyzing
9 dynamic problems associated with large deformation, low and high velocity impact,
10 ballistic penetration and wave propagation, etc. In particular, “contact algorithm” is
11 available in LS-DYNA, which enables more accurate simulation on multi-layer
12 materials like SHS composite by considering the interface performance.

13 **2.2 Material model**

14 In this numerical model, 6 types of materials needed to be modelled: the ECC
15 layer, HSC layer, AC layer, GST, steel bar and foundation soils. The first 3 materials
16 (ECC, HSC and AC) can be grouped as concrete material.

17 *2.2.1 Concrete damage model*

18 When subjected to blast loading or high impact loading, concrete or other
19 similar materials have shown a highly non-linear response. They usually exhibit
20 pressure hardening and strain hardening under static loading, and strain rate hardening
21 in tension and compression under dynamic loading. A number of material models have
22 been developed to model concrete recently (Holmquist *et al.* 1993, Malvar *et al.* 1997,
23 Polanco-Loria *et al.* 2008, Riedel *et al.* 2009). Among them, the MAT72 model has

1 been widely used to analyse concrete response to blast loading due to its simple
2 implementation. In addition, the MAT72 R3 model can capture the non-linear behaviour
3 of the material under dynamic loading (Malvar *et al.* 1997). Therefore, the MAT72 R3
4 was adopted in this study to simulate concrete and/or other similar materials including
5 AC, HSC, and ECC. The key features of the model are discussed briefly in the
6 following section.

7 ***Strength surface in MAT72 R3 model***

8 The MAT72 R3 model has three independent strength surfaces: “maximum
9 strength surface”, “yield surface” and “residual strength surface”, shown graphically in
10 Figure 1. During the initial increase of hydrostatic pressure P , the deviatoric stresses
11 $\Delta\sigma$ remains in the elastic region until the “yield surface” is reached. Deviatoric stress
12 can be further developed until the “maximum strength surface” is reached, and the
13 material will subsequently start to fail. After failure is initiated, the material will
14 gradually lose its load carrying capacity and reach its “residual strength surface”.

15 ***Damage factor in MAT72 R3 model***

16 After reaching the initial “yield surface” but before reaching the “maximum
17 strength surface”, the current surface can be obtained as a linear interpolation between
18 “yield surface” $\Delta\sigma_y$ and “maximum strength surface” $\Delta\sigma_m$:

$$19 \quad \Delta\sigma = \eta(\Delta\sigma_m - \Delta\sigma_y) + \Delta\sigma_y \quad (1)$$

20 After reaching the “maximum strength surface” the current failure is
21 interpolated between the “maximum strength surface” $\Delta\sigma_m$ and the “residual strength
22 surface” $\Delta\sigma_r$, which is similar to the above computation:

1
$$\Delta\sigma = \eta(\Delta\sigma_m - \Delta\sigma_r) + \Delta\sigma_r \quad (2)$$

2 where η varies from 0 to 1 depending on the accumulated effective plastic strain
 3 parameter λ . The value of η normally starts at 0 and increases to unity at $\lambda = \lambda_m$ and
 4 then decreases back to 0 at some larger value of λ . λ_m is the plastic strain at “maximum
 5 strength surface”. The accumulated effective plastic strain λ can be expressed as
 6 follows:

7
$$\lambda = \int_0^{\bar{\varepsilon}^p} \frac{d\bar{\varepsilon}^p}{r_f [1 + p / (r_f f_t)]^{b_1}} \quad \text{for } p \geq 0 \quad (3)$$

8
$$\lambda = \int_0^{\bar{\varepsilon}^p} \frac{d\bar{\varepsilon}^p}{r_f [1 + p / (r_f f_t)]^{b_2}} \quad \text{for } p < 0 \quad (4)$$

9 where f_t is the quasi-static tensile strength, $d\bar{\varepsilon}^p$ is effective plastic strain increment, r_f
 10 is the dynamic increase factor (DIF) of the material under dynamic loading. The
 11 damage factors b_1 and b_2 define the softening behaviour due to compression ($P \geq 0$)
 12 and tension ($P < 0$), respectively. Parameter b_1 can be determined by considering
 13 compressive energy G_c (area under compressive stress-strain curve) obtained from
 14 uniaxial compression test in single element simulation. It is obtained iteratively until the
 15 area under stress-stain curve from single element simulation coincides with G_c / h ,
 16 where h is the element size. Then b_2 is determined by considering fracture energy G_f
 17 from uniaxial tensile test or three points notched beam test in single element simulation.
 18 The value of b_2 is obtained until the area under tensile stress-stain curve from a single
 19 element coincides with G_f / w_c , where w_c is the localization width, and typically w_c is

1 taken as 1 to 6 times the maximum aggregate size (Malvar *et al.* 1997).

2 Based on Equations 2 and 4, the stress softening factors η and λ were governed
3 by the accumulation of effective plastic strain. However, when the stress path was very
4 close to the negative hydrostatic pressure axis, i.e. isotropic tension, wherein the
5 hydrostatic pressure would decrease from 0 to $-f_t$, where no deviatoric stress occurred,
6 no damage accumulation would have occurred based on these equations. However, in
7 such “concrete-like” materials damage could not be avoided even at this state.
8 Therefore, the above condition had to be modified by including pressure-softening
9 effects near or after tensile failures. In this case, a volumetric damage increment was
10 calculated and added to the total damage factor λ whenever the stress path was close to
11 the triaxial tensile path.

12 A scaled damage indicator δ was proposed to describe the damage level of the
13 material in this paper. The scaled damage indicator δ can be expressed as:

$$14 \quad \delta = \frac{2\lambda}{\lambda + \lambda_m} \quad (5)$$

15 in which, λ is accumulated effective plastic strain as defined in Equations 3 and 4.

16 It should be noted that there were three threshold values in Equation 5: (i) at
17 “yield surface”, $\lambda = 0$, leading to $\delta = 0$; (ii) at “maximum strength surface”, $\lambda = \lambda_m$,
18 leading to $\delta = 1$; and (iii) at “residual strength surface”, $\lambda = \lambda_r \square \lambda_m$, leading to
19 $\delta = 1.99 \approx 2$. Thus the varied δ value from 0 to 1 and then to 2 indicated that the
20 failure surface migrated from “yield surface” to “maximum strength surface” and then
21 to “residual strength surface” respectively, as the material was being stressed.

1 As this research focused on both the initiation and the degree of damage to the
2 “concrete-like” material subjected to blast loading, the post-peak behaviour was of great
3 interest. Thus, such post-peak behaviour obtained from FEM modelling was plotted for
4 the δ value from 1 to 2. The higher δ value represented the higher degree of damage.
5 In this study, it was further assumed that the threshold δ value classified for the
6 “severe crack” situation was 1.8, i.e. when δ value reached 1.8 and beyond, the
7 material was considered severely damaged.

8 *Strain rate effect*

9 The material model MAT 72 R3 also included a radial rate enhancement on the
10 material failure surface. This is because experimental data for “concrete-like” materials
11 were typically obtained along radial paths from the origin in deviatoric stresses versus
12 hydrostatic pressure via unconfined compressive and tensile tests.

13 *Equation of State (EOS)*

14 In addition to the strength surface model, an equation of state (EOS) is needed to
15 describe the relationship between hydrostatic pressure and volume change of the
16 material subject to dynamic load. EOS is usually determined using a fly impact (i.e. for
17 steel) test or triaxial compressive test (i.e. for concrete or geomaterials). The isotropic
18 compression portion of the MAT72 R3 material model consists of pairs of hydrostatic
19 pressure P and corresponding volume strain μ . It was implemented as a piece-wise
20 curve in this study.

21 *2.2.2 GST and Steel*

22 GST and steel were modelled using a “plastic-kinematic” model, a elastic-fully
23 plastic model with kinematic hardening plasticity, in accordance with Von Mises yield

1 criterion. The Von Mises yield criterion assumed that the initial yield or failure surface
2 was independent of the hydrostatic stress and the third invariant of the deviatoric stress
3 (Holmquist *et al.* 1993). Hence, it resulted in a circular shape with constant radius in
4 deviatoric plane and similar values for uniaxial yield tensile stress and uniaxial yield
5 compressive stress (Chen 1982).

6 *2.2.3 Foundation soil material*

7 The foundation soil was modelled using a “Drucker-Prager” model, in which the
8 cohesion and compaction behaviour resulted in an increasing resistance to shear until a
9 limit value of yield strength as the pressure increases (Chen 1982). Depending on the
10 matched stress states, the material constants are related to the constants c and ϕ of the
11 Mohr-Coulomb criterion in several ways (i.e. match along compressive meridian or
12 tensile meridian).

13 *2.3 Interface model*

14 The “interface model” was used for multi-layer composite systems, especially
15 between AC and HSC layer. The TIEBREAK contact algorithm was used to simulate
16 the interface behaviour between HSC and AC layers because it allowed simulation of
17 crack propagation at the interface. The stress-displacement response depends on the
18 energy release in the separation process of the interface. The simplest form of stress-
19 displacement had a linear elastic response until the crack initiation criterion was reached
20 and was then followed by a linear softening to zero stress when the damage was fully
21 reached (LS-DYNA 2007). The area under the stress-displacement curve was the
22 “energy released rate”, which described the energy dissipation during the development
23 of crack at interface.

1 The interface element was considered “failed” based on damage evolution.
2 Damage is defined as a linear function of the distance between points initially in
3 contact. When the distance exceeds the defined critical distance, the interface is
4 considered completely failed. Thus, the energy release rates G_I and G_{II} for normal and
5 shear interface failure modes were defined as:

$$6 \qquad G_I = \frac{1}{2} \tau_n \square D \qquad (6)$$

$$7 \qquad G_{II} = \frac{1}{2} \tau_s \square D \qquad (7)$$

8 where, τ_n and τ_s are the normal stress and shear stress at the interface, D is the critical
9 displacement for total failure.

10 Tensile (pullout test) and direct shear tests were conducted to obtain the energy
11 release rate G_I and G_{II} . It should be noted that the value τ_n and τ_s was related to the
12 characteristic element length (square root of area at interface). Usually, the low failure
13 stress value was needed for coarser meshes. Hence, τ_s and D could first be numerically
14 determined by matching the load-displacement curve from the direct shear test, and then
15 τ_n could be obtained by the known value of D and G_I via Equation 6. After the failure
16 criterion was met, the nodes were separated. The interface then behaved the same as the
17 surface-to-surface contact type which transferred the shear stress and compressive stress
18 at the interface.

19 **3. DETERMINATION OF MATERIAL MODEL PARAMETERS**

20 This section summarises the key parameters for the material models in
21 modelling the innovative SHS multi-layer pavement composite under blast loading.

1 **3.1 AC layer**

2 The MAT72 R3 model was employed to simulate AC to capture post-peak
3 behaviour. This model did not consider the temperature effect. However, during the
4 blast event, the temperature suddenly increased to a thousand degrees in a few
5 microseconds, and then dropped quickly with propagation distance. Based on the field
6 test, only the central part of the AC layer was destroyed under the combined effects of
7 high temperature and blast pressure. With the increase in distance from the centre, the
8 failure of the central AC surface was mainly due to blast pressure, and the high
9 temperature effect could be ignored. Therefore, the MAT72 R3 material model for AC
10 was used in this research.

11 **3.1.1 Strength Surface**

12 As mentioned in section 2.2, the MAT72 R3 material model had three strength
13 surfaces: strength, residual strength, and yield surfaces. These three surfaces can be
14 obtained through curve fitting of suitable experimental data. Available data was
15 extracted from Park *et al.* 2005 with the compressive strength $f_c=0.311$ MPa for AC.
16 Figure 2 presents the determination of the three surfaces by curve fitting for AC with
17 $f_c=0.311$ MPa. The intersection point of maximum strength surface and residual strength
18 surface was the brittle-to-ductile point. This point should be determined by
19 experimental data under high confining pressure. However, it was difficult to determine
20 this point in strength surface as no experimental data was available for AC. Based on
21 the experimental data for concrete, this point was usually taken as $p/f_c=3.878$.
22 Considering the size and strength of aggregates used in AC and concrete was almost the
23 same, the brittle-to-ductile point for AC was taken to be the same as that for concrete.
24 This value may be conservative for AC due to the higher content of coarse aggregate

1 mixed in asphalt mixture. However, in terms of the simulation results, this value could
2 be acceptable. In this study, the unconfined compressive strength for AC in field trial
3 test was 4.6 MPa, and the tensile strength was 0.7 MPa at 35°C. By scaling the data
4 from the established curves given in Figure 2, the appropriate strength surface of the
5 current materials can be determined as shown in Figure 3.

6 *3.1.2 Damage factor*

7 The strain hardening and softening pairs (η, λ) in Equations 1 and 2 described
8 the material behaviour transmitted from “yield surface” to “maximum strength surface”
9 and from “maximum strength surface” to “residual strength surface”, respectively.
10 During the transmission, parameter η varied from 0 to 1 depending on the accumulated
11 effective plastic strain parameter λ . However, it was found that the original damage
12 factor pairs (η, λ) in the MAT72 R3 model were only suitable for concrete and not for
13 the AC material due to AC having higher plastic failure strain. Thus, the input for
14 accumulated effective plastic strain λ was modified. Based on the uniaxial compressive
15 test for AC, it was found that at peak stress the corresponding strain was approximately
16 0.018 and the final failure strain was about 0.1. While for normal concrete, the
17 corresponding strain at peak stress was around 0.0022. Hence, the λ was modified to
18 give the high failure strain for AC. Additionally, it was found that when λ was adjusted
19 to 10 times the original λ value the numerical results seemed to show good agreement
20 with experimental results from unconfined compressive test for AC. Figure 4 shows the
21 modified and original series of (η, λ) pairs. It can be seen that the modified damage
22 factor made smoother descending than the original damage factor, and had a higher
23 failure strain that seemed to match the behaviour of AC very well.

1 3.1.3 Equation of state

2 There were limited EOS data for AC. The available EOS data is for AC with
3 compressive strength of $f_c=3.8$ MPa (Tang *et al.* 2009). In this study, the compressive
4 strength for AC was $f_c=4.6$ MPa. Thus, the pressure-volume pairs were calculated using
5 the scaling method (Malvar *et al.* 1996). In this method, assuming that new data would
6 be obtained at the same volumetric strains, and thus the new corresponding pressure (p_{new}) would be:

$$8 \quad p_{new} = p_{old} \sqrt{r} \quad (8)$$

9 and the new corresponding unloading bulk modulus (ku_{new}) would be:

$$10 \quad ku_{new} = ku_{old} \sqrt{r} \quad (9)$$

11 where r is the scaling factor which is the ratio of compression strength for new material
12 to compression strength of the previous material modelled. Hence, the EOS data for
13 $f_c=4.6$ MPa was calculated based on Equations 8 and 9 as shown in Figure 5.

14 3.1.4 Softening parameter b_1, b_2

15 The softening parameters (b_1, b_2) shown in Equations 3 and 4 controlled the
16 material softening behaviour after peak stress. These parameters were obtained from
17 experiments, as detailed below.

18 A) Value of b_1 from uniaxial compressive test

19 The uniaxial compressive test was conducted for AC according to ASTM 1074.
20 The compression testing results are presented in Figure 6. It was found that the

1 corresponding strain at peak stress ($f_c=4.6$ MPa) was about 0.018 and the final failure
2 strain was about 0.1, which was higher than that of concrete. The Young's modulus
3 obtained from strain gauges attached at the middle height of the sample was 598 MPa.
4 Based on experimental results, the compressive energy G_c was calculated at 15.1
5 MPa·mm. Hence, b_1 for different element sizes were obtained through single element
6 simulation as suggested in section 3.1. The b_1 value for 10 mm mesh size was found to
7 be 4.2.

8 B) Value of b_2 from fractural test

9 The value of b_2 was determined by fracture energy G_f , which can be obtained
10 from uniaxial tensile test or three points Single-edge Notched Beam test (SNB). In the
11 current study, the SNB test was carried out to evaluate fracture energy G_f for AC.
12 Detailed theory about the SNB test can be found in established literature (Karihaloo and
13 Nallathambi 1990). Therefore, only the test result is presented here. In the SNB test, the
14 compacted AC beam was fabricated with a dimension of $400 \times 100 \times 100$ mm³ depth. A
15 mechanical notch was sawn with a depth of 20 mm, which gave a ratio of notch to beam
16 depth of 0.2. The simply supported sample with a span length of 340 mm was tested
17 under 35°C temperature. From the test, fracture toughness K_{IC} was obtained according
18 to the formula suggested by Karihaloo and Nallathambi (1990). Then the fracture
19 energy G_f was calculated using:

$$20 \quad G_f = \frac{(1-\nu^2)K_{IC}^2}{E} \quad (10)$$

21 in which E is the elastic modulus and ν is poisson's ratio.

1 b_2 was obtained via iterative procedure until the area under stress-stain curve
 2 from single element simulation coincided with G_f / w_c . The parameters obtained from
 3 SNB and single element simulation for AC ($f_c=4.6$ MPa) are summarized in Table 1.

4 3.1.5 Strain rate effect for asphalt material

5 Dynamic increase factor for compression

6 In the current study, DIF for AC under different strain rate was obtained using
 7 servo hydraulic fast loading tests and the split Hopkinson pressure bar (SHPB) test. The
 8 strain rate produced by the servo hydraulic machine was approximately 10^{-5} to 1 s^{-1} , and
 9 the higher strain rate loading was obtained through SHPB testing.

10 The DIFs for AC under different strain rate from SHPB and the servo hydraulic
 11 machine test is illustrated in Figure 7. It shows that the DIF increased with the increase
 12 of strain rate. Moreover, the DIF value increased sharply at a certain strain rate, which
 13 was similar to the behaviour of the “concrete-like” material. The fitted curve of the
 14 AC’s DIF data had two segments, with a threshold point at $100s^{-1}$. For strain rate less
 15 than $100s^{-1}$, it was in a monotonic increasing trend, while it was a straight line with
 16 strain rate exceeding $100s^{-1}$. Hence, the dependence of DIF on strain rate for AC under
 17 compression ($DIF_{AC,c}$) was proposed as following piece-wise functions:

$$18 \quad DIF_{AC,c} = \frac{f_d}{f_s} = 3.18 + 1.098 \log_{10}(\dot{\epsilon}) + 0.1397 \log_{10}^2(\dot{\epsilon}) \quad \text{for } \dot{\epsilon} \leq 100s^{-1} \quad (11)$$

$$19 \quad DIF_{AC,c} = \frac{f_d}{f_s} = 21.39 \log_{10}(\dot{\epsilon}) - 36.76 \quad \text{for } 100s^{-1} < \dot{\epsilon} \leq 200s^{-1} \quad (12)$$

20 A numerical modelling of the SHPB test adopting this DIF curve found that the
 21 initial segment of this curve matched the experimental results very well, while the

1 numerical model results for strain rate larger than 100s^{-1} seemed to overestimate the
2 stress. This could be due to the “double counting” of the inertia effect in the numerical
3 modelling when strain rate exceeded 100s^{-1} . Hence, in the current model, the second
4 segment was ignored when the strain rate exceeded 100s^{-1} . Beyond this, the DIF was
5 assumed to remain a constant value.

6 *Dynamic increase factor for tension*

7 For the high strain rate in tension, the SHPB setting was employed for
8 conducting the dynamic splitting tensile test. The servo hydraulic machine was also
9 used to conduct dynamic splitting tensile testing for a low to moderate strain rate range.
10 The configuration of the test and sample information can be found in Wu (2012). The
11 test results are presented and discussed here. The test results for dynamic tensile
12 strength of AC are shown in Figure 8. From the figure, it can be seen that the splitting
13 tensile strength increased with the increase of the strain rates following a bi-linear trend.
14 The transition point was found to be at 15s^{-1} . The post-experimental picture revealed
15 that the binder failure and trans-aggregate failure had occurred at about 15 to 80s^{-1} ,
16 consistent with observations from Tekalur *et al.*(2009). The stress wave within the
17 specimen went through the aggregated or binder material under dynamic loading while
18 under the static loading, the failure usually occurred at the weakest component
19 (interfacial zone) within the specimen. The dependence of DIF on strain rate for AC
20 under tension ($DIF_{AC,t}$) was proposed based on experimental results as following piece-
21 wise functions:

$$22 \quad DIF_{AC,t} = \frac{f_d}{f_s} = 1.86 + 0.1432 \log_{10}(\dot{\epsilon}) \quad \text{for } \dot{\epsilon} \leq 15\text{s}^{-1} \quad (13)$$

1
$$DIF_{AC,t} = \frac{f_d}{f_s} = 6.06 \log_{10}(\dot{\epsilon}) - 5.024 \quad \text{for } 15s^{-1} \leq \dot{\epsilon} \leq 100s^{-1} \quad (14)$$

2 In the macro-level numerical model, the MAT 72R3 material model cannot
3 capture the aggregate interlocking that propagates the micro-cracking and energy
4 dissipation beyond the localization zone (Magallanes *et al.* 2010). Therefore, the above
5 proposed tensile DIF curve with two branches was used in the model.

6 **3.2 HSC and ECC layer**

7 The MAT72 R3 model was also used to simulate HSC and ECC. The parameters
8 for HSC are illustrated in Table 2. For HSC, the equation recommended by CEB
9 (Comite Euro-International du Beton 1993) was adopted to consider the dependence of
10 DIF on strain rate. However, similar to the above-mentioned AC material, only the first
11 segment of the compressive DIF curve for concrete-like material was employed in the
12 present study. The tensile and compressive DIF curves used for the numerical model are
13 shown in Figure 9.

14 The ECC was first simulated by Lee (2006) using the MAT 72 R3 model, and it
15 showed that such a material model was suitable for modelling ECC material under
16 dynamic loading, including impact and blast loading. Hence, the MAT72 R3 was used
17 to simulate ECC in this study. The material properties of ECC are given in Table 2. The
18 DIF curve for ECC under compression and tension were recommended by Lee (2006)
19 and Maalej *et al.* (2005), respectively. Both tensile and compressive DIF curves for
20 ECC used in the model are shown in Figure 10.

21 **3.3 GST and Steel**

22 A) GST

1 Geosynthetics reinforcement may enhance the tensile strength of the AC layer,
2 Therefore, it is necessary to consider the function of the GST material. In the current
3 study, the GST reinforcements were simulated with shell element formulation in LS-
4 DYNA due to its computational efficiency. The thickness of the shell element was taken
5 as the average between the rib and the junction thickness (Lee *et al.* 2010). Since the
6 GST showed the bilinear stress-strain behaviour, where some hardening behaviour had
7 taken place after the initial yield point P as shown in Figure 11. The plastic-kinematic
8 model was employed to simulate the behaviour of GST. The parameters for GST in the
9 plastic-kinematic model were determined by fitting the bilinear curve with experimental
10 load-strain curve, shown in Figure 11. The parameters for GST used in this simulation
11 are summarized in Table 3.

12 B) Steel bar

13 A steel bar is an isotropic material having the same initial yield stress for both
14 uniaxial tension and uniaxial compression. The plastic-kinematic model was suitable to
15 model isotropic and kinematic hardening plasticity. Thus, the plastic-kinematic model
16 was employed to describe the behaviour of the steel bar in this study. The material
17 parameters of the steel bar in this study are summarized in Table 3.

18 **3.4 Foundation soil**

19 The Drucker-Prager model was used to model the subgrade soil foundation. The
20 soil parameters in the model are estimated from actual soil investigation performed on
21 the test site (Wang *et al.* 2010) as shown in Table 4.

1 **3.5 Interface properties**

2 For the SHS composite, it was assumed that the HSC and ECC were fully
3 bonded, while the interface between HSC and AC was weaker than that between HSC
4 and ECC. This was mainly due to the construction method's difference as the AC layer
5 was directly placed on the HSC layer a few days after HSC casting. Strain and stress
6 can be transferred between AC and HSC when vertical and horizontal loading is applied
7 on the AC layer. The interfacial shear strength between AC and HSC can influence the
8 integrity of the stress/strain transfer. Therefore, laboratory investigations were
9 conducted to obtain interface properties between the HSC and AC layers.

10 *3.5.1 Shear strength and static friction coefficient*

11 A direct shear test method was adopted to investigate the shear behaviour
12 between AC and HSC. The configuration of the test is presented in Figure 12. The
13 direct shear test was conducted under constant vertical loading and four levels of
14 vertical loading were applied: 2.1 MPa, 1.5 MPa, 1 MPa and 0.5 MPa. The maximum
15 tire pressure -2.1MPa from typical military and civilian aircraft was set as the maximum
16 vertical loading in the direct shear test.

17 The results from the direct shear test are shown in Figure 13. It clearly shows
18 that the shear strength for interface was around 1.5 MPa for the maximum normal
19 loading 2.1 MPa. This value was set as the maximum shear strength for the current
20 interface between AC and HSC. At the failure surface, it was found that the asphalt
21 surface was smashed during the shear test, while the HSC surface had less damaged
22 than that of asphalt surface. Shear failure was mainly due to the failure of the AC
23 surface. It was found that the static friction coefficient of the AC and HSC interface was
24 0.71, or a friction angle of 35°.

1 3.5.2 Dynamic friction coefficient

2 Dynamic friction is defined as the frictional force between two moving solid
3 surfaces in contact with each other. Where the objects are in motion, there will still be
4 frictional force. Usually, the dynamic friction coefficient was lower than the static
5 friction coefficient. A tilt table test was used to evaluate the dynamic friction coefficient.
6 The theory of the tilt table test is shown in Figure 14. In the figure, it is assumed that the
7 solid body was sliding along the tilt surface with certain acceleration. The weight of
8 solid body is mg , the tilt angle from the horizontal surface is θ . For the solid body, the
9 force along tilt surface from its own weight is expressed as $mg \sin \theta$, the component
10 normal to the tilt surface is $mg \cos \theta$. Considering the equilibrium of force normal to the
11 tilt surface, the reaction force R equals to $mg \cos \theta$. Thus the dynamic friction F is
12 expressed as $\mu_{dynamic} mg \cos \theta$. The total sliding force along the tilt surface is then
13 expressed as $(mg \sin \theta - \mu_{dynamic} mg \cos \theta)$. Hence, according to the Newton's second law
14 $F = ma$, the acceleration of solid body can be expressed as:

15

$$16 \quad a = g(\sin \theta - \mu_{dynamic} \cos \theta) \quad (15)$$

17 If the acceleration is zero, meaning that the solid body is sliding along the tilt
18 surface with constant velocity, based on the Equation 5.1, the dynamic friction
19 coefficient can be resolved as:

20

$$21 \quad \mu_{dynamic} = \frac{\sin \theta}{\cos \theta} = \tan \theta \quad (16)$$

22 It can be seen that the dynamic friction coefficient can be directly obtained from
23 the tilt angle, when the rigid body object is sliding at a constant speed down the slope.

1 The set-up to evaluate the dynamic friction coefficient between the HSC and AC
2 layers is shown in Figure 15. The AC block was placed along the tilt surface, made of
3 HSC material. The tilt angle was adjusted to make the AC block slide along the surface
4 with constant velocity, from which the friction coefficient was derived. With a few
5 tests, the dynamic friction angle between AC and HSC layer was found to be about 29°
6 to 30°. Hence, the dynamic friction coefficient was between 0.55-0.57. According to the
7 result of the direct shear test, the static friction coefficient between AC and HSC was
8 around 0.71, which corresponded to the friction angle of 35°. It was found that the
9 dynamic friction coefficient was about 78% of the static friction coefficient. In the
10 current study, the dynamic friction coefficient is taken as 0.56.

11 *3.5.3 Interface model validation*

12 The simulation of the direct shear test using TIEBREAK contact was validated
13 using the direct shear test conducted above. The size of the model was taken as the same
14 as that in the laboratory test. Solid element was used to model AC and HSC material.
15 The TIEBREAK contact algorithm was employed to simulate the interface behaviour.
16 The parameters for interface simulation are listed in Table 5. The load-displacement
17 curve obtained from the simulation was compared with the experimental result and it
18 was found that both peak shear force and the corresponding displacement at peak shear
19 force from the numerical model agreed well with that obtained from the experiments.
20 Therefore, the TIEBREAK contact in LS-DYNA was confirmed as capable of
21 modelling the interface behaviour of HSC and AC and was used for the 3D simulation.

22 **4. FULL SCALE BLAST TEST**

23 One SHS multilayer pavement composite slab was designed and tested in the
24 field under blast loading. A bomb the equivalent of a 7.3kg TNT charge weight was

1 placed at the centre of the slab, with the bomb's centre of gravity elevated at 170 mm
2 above the slab surface. This multi-layer pavement slab was cast on site with a
3 dimension of $2.8 \times 2.8 \times 0.275 \text{ m}^3$. Figure 16 shows the cross-sectional view of this slab.
4 A 100mm thick ECC layer was placed at the bottom of the slab. Another 100mm thick
5 HSC layer was constructed above the ECC layer before it was completely hardened in
6 order to improve the interface bonding. These two bottom layers were allowed to cure
7 for one week, and then the third AC layer was cast on the top of the existing two layers.
8 The AC layer was cast in two sub-layers with a thickness of 38mm for each. Both layers
9 were compacted using a small 1-tonne compactor. A GST layer was placed in-between
10 the two asphalt sub-layers as reinforcement for the AC layer. The GST was Polyfelt
11 Microgrid MG-100 with bi-directional tensile strength at 100 kN/m and had an aperture
12 size of 7 mm.

13 To facilitate the lifting and transportation of the completed slabs to the site,
14 minimal reinforcements -T12 steel bars in both directions at about 350mm spacing, with
15 a cover of 25mm were installed in the bottom ECC layer. This reinforcement was
16 undertaken to prevent the pavement slab from cracking under its own weight during
17 transportation. To simulate an actual pavement condition, which is much wider and with
18 sufficient side anchorage, the pavement slab was anchored to the ground. Thus, four
19 vertical anchors were installed near the corner of the pavement slab. Figure 17 shows a
20 photograph of the completed pavement slab with four anchors.

21 Various instruments were installed onto the slab to measure its responses during
22 blast loading. Figure 18 shows the instrumentation installed on the pavement slab. Four
23 accelerometers were installed at the middle of the side of the slab to measure both
24 vertical (V1 and V2 in Figure 18) and horizontal accelerations (H1 and H2 in Figure
25 18). The accelerometers were mounted onto steel frames that were cast together with

1 the slab. Three total pressure cells (TPC) (TPC1, TPC 2 and TPC3 in Figure 18) were
2 buried in the soil just below the slab to measure the pressure transferred from the
3 pavement slab. The measurement results of the field blast testing will be discussed and
4 compared with the numerical simulation results in section 5.

5 **5. MODELLING SHS COMPOSITE UNDER BLAST LOADING**

6 ***5.1 Model development***

7 The multi-layer slab and soil mass were discretized in space with one point
8 gauss integration eight-node hexahedron Lagrange element. Only a quarter of the slab
9 was modelled considering symmetry. Thus, the dimension of the slab in the numerical
10 model was $1400 \times 1400 \times 275 \text{ mm}^3$. Regarding the soil domain, with mesh size
11 convergence analysis, the thickness and length of soil mass were taken as 5600 mm and
12 7000 mm, respectively.

13 The reinforcement bars and GST were spatially discretized using beam element
14 and shell element, respectively. It was assumed that the reinforcement bars and GST
15 were fully bonded within the ECC and AC (in the field blast test, the rebar was placed
16 in the layer of ECC, and the GST was placed in the AC layer). The anchors on the
17 multi-layer pavement slab were also considered and simulated as fixed points in the
18 corresponding position in the numerical model. The soil mass was treated as a semi-
19 infinite space. Thus, the non-reflection boundary was applied on the side and bottom of
20 the soil mass. The Automatic_Surface_To_Surface contact algorithm was employed to
21 simulate the interaction between the pavement slab and the soil mass.

22 The element cells for the pavement slab had an aspect ratio of 1, suitable to
23 simulate wave propagation. Due to the computational time and capability, “bias mesh
24 technology” was adopted for the soil mass. In the central part of the soil mass under the

1 pavement slab, the mesh size was uniform with an aspect ratio of 1. The mesh size was
2 gradually increased away from the centre part of the soil mass, both radially and
3 vertically. After the convergence study, it was found that a 10 mm element size in the
4 centre part of the slab gave stable and accurate results.

5 LS-DYNA uses a commonly used blast load routine CONWEP (Hyde 1992) to
6 generate blast loadings. It should be noticed that the blast pressure from CONWEP were
7 well calibrated using many full scale field tests by the US Army. The minimum scaled
8 distance (scale distance = [distance between bomb and target] / [TNT charge weight of
9 bomb]^{1/3}) in CONWEP is around 0.15 m/kg^{1/3}, meaning that the blast pressure would be
10 accurate when the actual stand-off scaled distance is larger than this threshold value.
11 This can be classified as a middle to far field blast range. However, when the scaled
12 distance is smaller than this value (i.e. close-in blast range), the blast pressure generated
13 in CONWEP is obtained through extrapolation from the blast pressure at 0.15 m/kg^{1/3}
14 scaled distances, which may not be accurate enough. In this study, the charge weight of
15 7.3 kg equivalent TNT was placed 170 mm above the slab. Thus, the scaled distance
16 was 0.087 m/kg^{1/3}, and the blast pressure obtained from the CONWEP model might not
17 be accurate. Furthermore, the CONWEP method adopted the planar blast wave front
18 when the blast wave reached the target, which means that the whole target was
19 subjected to a uniform magnitude of blast pressure. This simplified method is
20 acceptable when the target is relatively small. However, in the current study, the target
21 was a large piece of pavement slab. During the blast event, the blast pressure exerted on
22 the slab would be a function of distance and time of arrival, therefore, to more
23 accurately model such blast pressure, a 3D computational fluid dynamic (CFD) program
24 called AUTODYN was used in this study. The blast pressure was extracted from
25 AUTODYN and then applied onto the slab model target in LS-DYNA. The detailed

1 process of applying pressure to the pavement surface can be found in Wu and Chew
2 (2014).

3 **5.2 Simulation results**

4 The results of the numerical modelling of the SHS pavement composite under
5 blast loading, with the incorporation of the above-mentioned material models, are
6 summarized and discussed in this section. The simulation results are compared with the
7 blast test results and discussed below.

8 The damage situation for new multi-layer pavement slab in field blast test is
9 shown in Figures 19 and 20. Figure 19 shows that the blast pressure destroyed the upper
10 half of the asphalt layer above the GST reinforcement. It was also noted that only the
11 centre of the GST piece was burned off during the blast event. Figure 20 shows the
12 resulting damage on the HSC layer after removing the top layer of asphalt. From this
13 figure, it can be seen that the crater was very shallow and did not punch through the
14 whole layer and a crater of around 700mm diameter and depth of 10mm was formed on
15 the HSC layer.

16 The results of the damage situation for new multi-layer pavement slab using the
17 numerical model are given in Figure 21. The “fringe level” in the contour is the value
18 for scaled damage indicator δ , which is defined in Equation 5. When the δ value is
19 greater than 1.8, the material is considered severely damaged. The damage pattern for
20 AC layer is shown in Figure 21(a). Comparing Figure 21(a) and Figure 19, it is
21 observed that the damage pattern in the numerical model is symmetrical while that in
22 the field measurement is skewed. This is because the bomb in the field was not placed at
23 the centre of the slab, and one side of the asphalt was more severely damaged than the
24 other. Shear cracking near the anchor point was observed in the numerical model, which

1 was similar to the experimental observations in the field test. It could be concluded that
2 the basic failure pattern given by the numerical model agreed well with the results
3 obtained from the field-testing.

4 Figure 21(b) shows the damage pattern for the HSC layer. Comparing Figure
5 21(b) with Figure 20, the damage pattern for HSC was very consistent between field
6 measurement and numerical results. The diameter of the crater was about 750mm in the
7 numerical model, which was quiet close to that of the blast test result. As shown in
8 Figure 21(b), shear cracks were also observed near the anchor points. Based on the
9 damage pattern in the field blast test, the crater on the top face of the HSC was shown to
10 be shallow and with a thickness of less than 10mm. However, after cracking occurred at
11 the bottom face of the HSC layer, the numerical model showed that the bottom of the
12 HSC had experienced severe cracking. This might be due to the combination of the
13 bending of the HSC layer under blast load and the reflection of the stress wave at the
14 bottom interface. In the numerical model, the interface between HSC and ECC was
15 assumed to be fully bonded. However, ECC is more flexible than HSC, and thus it
16 would cause tensile stress at the bottom face of HSC layer when deformed together. The
17 compression stress wave from the top face would also travel within the HSC layer and
18 reflect as a tension stress at the interface, which could cause spalling. Based on the
19 damage pattern in the numerical model, the HSC layer may be considered failed, while
20 the field observation may suggest that HSC was partially failed.

21 Figure 21 (c) shows the damage pattern the for ECC layer. As shown in the
22 figure, only small amounts of moderate cracks were found at the centre of the top face
23 of the ECC layer. Some severe cracks were found at the bottom face but all within a
24 very limited area. Field observation of the ECC layer showed that it was still intact and
25 functional.

1 Above all, in terms of the damage pattern, the behaviour of the SHS composite
2 under blast loading was well predicted by the numerical model. For this multi-layer
3 composite, only the top AC layer required major repair or replacement, while the HSC
4 layer needed minor repair. The bottom ECC layer can be considered undamaged. This
5 indicates the advantage of using the proposed multi-layer system.

6 In the blast testing, four accelerometers were installed at the mid-side of the
7 pavement slab to measure the vertical and horizontal acceleration of the pavement slab
8 subjected to blast loading. For the horizontal acceleration, the horizontal acceleration
9 readings on the two sides were not the same due to the centre of the charge (conical
10 shape) being closer to one side of the pavement slab. While in the numerical model, it
11 was assumed that the explosion occurred in the centre of the pavement slab. Thus, in
12 this section, only the vertical acceleration from the field blast test was compared with
13 that of the numerical model.

14 The vertical acceleration from the blast testing was compared with that of
15 numerical model as shown in Table 6. The results from both the blast testing and the
16 numerical simulation were comparable. The maximum difference of vertical
17 acceleration between the blast testing and the numerical model was about 10%, and the
18 numerical model predicted slightly higher in vertical acceleration than that of the blast
19 test.

20 The pressure values in the corresponding points in the numerical model were
21 compared with pressures obtained from the blast test, as summarized in Table 7. The
22 pressure values from the numerical simulation were shown to be close to that from the
23 blast test for TPC2. While for TPC3, it showed a 20% discrepancy, which was
24 acceptable in the numerical simulation considering the inherent variation in the blast

1 test.

2 TPC1 was damaged during the blast test, and no pressure reading was recorded
3 from it. The numerical model predicted the pressure might be as high as 13 MPa at that
4 point which was far beyond the maximum measurement capacity of the pressure cell
5 installed. That could explain why TPC1 was destroyed due to the overwhelming blast
6 loading.

7 **6. CONCLUSIONS**

8 A 3D numerical model was developed to model the behaviour of the innovative
9 SHS multi-layer pavement composite under blast loads. The SHS composite included
10 three layers; one layer of GST reinforced asphalt on the top, one layer of engineered
11 cementitious composites (ECC) at the bottom, and another layer of high strength
12 concrete (HSC) in between. The concrete damage model in LS-DYNA was adopted for
13 modelling. The modified damage factor proposed in this paper made smoother
14 descending than the original damage factor, and had a higher failure strain that could
15 improve the simulation of the behaviour of AC. Contact algorithm was incorporated to
16 simulate the behaviour of the interfaces. The key parameters for the material models
17 were all verified according to a variety of experiments. In particular, interface properties
18 between HSC and AC were considered and the model was validated through laboratory
19 experiments. Moreover, the strain rate effects were also added into the model. Modified
20 compressive and tensile DIFs for asphalt concrete were also proposed with
21 consideration of the strain rate effects, based on the results obtained from the SHPB
22 tests.

23 Using the developed model, the performance of the innovative SHS composite
24 subjected to blast loading was simulated and compared to the corresponding blast test

1 rests. The damage pattern of the SHS composite under blast loading compared with that
2 obtained in the blast test was well predicted by the numerical model. The vertical
3 acceleration response from the numerical simulation was also comparable to that from
4 the blast testing. In addition, the pressure values from the numerical model were also
5 reasonably consistent with the pressures measured from the blast tests though the
6 numerical model predicted slightly higher than that from the blast test. In addition, Both
7 the numerical and field blast test results showed that the SHS composite exhibited high
8 resistance against blast loading, as per its design.

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14 **REFERENCES**

- 15 Chen, W.F. (1982). *Constitutive equations for engineering materials*: John Wiley & Sons.
16 Comite Euro-International Du Beton, 1993. Cep-fip model code 1990. Redwood Books, Trowbridge,
17 Wiltshire, UK.
18 Dancygier, A.N. & Yankelevsky, D.Z. (1996). High strength concrete response to hard projectile impact.
19 *International journal of Impact Engineering*, 18 (6), 583-599.
20 Holmquist, T.J., Johnson, G.R. & Cook, W.H. (1993). A computational constitutive model for concrete
21 subjected to large strains, high strain rates, and high pressures. *The 14th International Symposium on*
22 *Ballistics*. Quebec, Canada, 591-600.
23 Hyde, D. (1992). *Conwep-application of tm5-855-1. Fundamentals of protective design for conventional*
24 *weapons*. Vicksburg, MS.
25 Karihaloo, B.L. & Nallathambi, P. (1990). Effective crack model for the determination of fracture
26 toughness (k_{ic}^c) of concrete. *Engineering Fracture Mechanics*, 35 (4/5), 637-645.
27 Koerner, R.M., 1998. *Designing with geosynthetics* N.J.: Prentice-Hall Eaglewood.
28 Lee, K.Z.Z., Chang, N.Y. & Ko, H.Y. (2010). Numerical simulation of geosynthetic-reinforced soil wall
29 under seismic shaking. *Geotextiles and Geomenbranes*, 28, 317-334.
30 Lee, S.C. (2006). *Finite element modeling of hybrid-fiber ecc targets subjected to impact and blast[d]*.
31 National University of Singapore.
32 Li, V.C. & Maalej, M. (1996). Toughening in cement based composites-part ii: Fiber-reinforced
33 cementitious composites. *Journal of Cement and Concrete Composites*, 18 (4), 239-249.
34 Li, V.C., Mishra, D.K., Naaman, A.E., Wight, J.K., Wu.H.C. & Inada, Y. (1994). On the shear behavior
35 of engineered cementitious composites. *Journal of Advanced Cement Based Materials*, 1 (3), 142-149.
36 LS-DYNA (2007). *LS-DYNA keyword user's manual*: Livermore Software Technology Corporation
37 (LSTC).

1 Maalej, M., Quek, S.T. & Zhang, J. (2005). Behavior of hybrid-fiber engineered cementitious composites
2 subjected to dynamic tensile loading and projectile impact. *Journal of Materials in Civil Engineering*,
3 17 (2), 143-152.

4 Magallanes, J.M., Wu, Y., Malvar, L.J. & Crawford, J.E. (2010). Recent improvements to release iii of
5 the k&c concrete model. *11th International LS-DYNA Users Conference*. Detroit, USA, 37-48.

6 Malvar, L.J., Crawford, J.E. & Wesevich, J.W. (1996). *A new concrete material model for dyna3d release*
7 *ii: Shear dilation and directional rate enhancements*. Defense Nuclear Agency.

8 Malvar, L.J., Crawford, J.E., Wesevich, J.W. & Simons, D., 1997. A plasticity concrete material model
9 for dyna3d. *International Journal of Impact Engineering*, 19 (9-10), 847-873

10 Park, D.W., Martin, T., Lee, H.S. & Masad, E. (2005). Characterization of permanent deformation of an
11 asphalt mixture using a mechanistic approach. *KSCE Journal of Civil Engineering*, 9 (3), 213-218.

12 Polanco-Loria, M., Hopperstad, O.S., Borvik, T. & Berstad, T. (2008). Numerical predictions of ballistic
13 limit for concrete slabs using a modified version of the hjc concrete model. *International Journal of*
14 *Impact Engineering*, 35, 290-303.

15 Riedel, W., Kawai, N. & Kondo, K. (2009). Numerical assessment for impact strength measurements in
16 concrete materials. *International journal of Impact Engineering*, 36, 283-293.

17 Tang, W.-H., Ding, Y.-Q. & Yuan, X.-Y. (2009). The hjc model parameters of an asphalt mixture.
18 *DYMAT 2009 - 9th International Conference on the Mechanical and Physical Behaviour of Materials*
19 *under Dynamic Loading*. 1419-1423.

20 Tekalur, S.A., Shukla, A., Sadd, M. & Lee, K.W. (2009). Mechanical characterization of a bituminous
21 mix under quasi-static and high-strain rate loading. *Construction and Building Materials*, 23, 1795-
22 1802.

23 Wang, F., Lim, C.H. & Soh, T.B. (2010). Explosive testing, numerical and analytical modelling of a
24 modular blast wall system. *The 3rd International Conference on Design and Analysis of Protective*
25 *Structures*. Singapore, 392-401.

26 Wu, J. (2012). *Development of advanced pavement materials system for blast load[d]*. National
27 University of Singapore.

28 Wu, J. & Chew, S.H. (2014). Field performance and numerical modelling of multi-layer pavement system
29 subject to blast load. *Construction and Building Materials*, 52, 177-188.

30 Zhang, M.H., Sharif, M.S.H. & Lu, G., 2007. Impact resistance of high strength fibre-reinforced concrete.
31 *Magaine of Concrete Research*. 199-210.

32 Zhang, M.H., Shim, V., P.W, Lu, G. & Chew, C.W. (2005). Resistance of high-strength concrete to
33 projectile impact. *International Journal of Impact Engineering*, 31, 825-841.

34

35

Table 1. Parameters from SNB and single element simulation

Parameters	Unit	Value
K_{IC}	MPa • mm ^{1/2}	12.2
ν	---	0.35
E	MPa	598
G_f	MPa • mm	0.221
w_c	mm	40
G_f/w_c	---	0.00554
f_t	MPa	0.7
b_2	---	0.2

Table 2. Material properties of HSC and ECC

Parameters	Units	HSC	ECC
Young's modulus, E	GPa	33	18
Compressive strength, f_c	MPa	55	64
Tensile strength, f_t	MPa	4.35	5
Poisson's ratio, ν	---	0.2	0.22
Density, ρ	kg/m ³	2400	2080

Table 3. Parameters for geosynthetics MG-100 and steel materials using a plastic-kinematic model

Parameters	Units	GST	Steel
Young's modulus, E	MPa	500	207000
Yield stress, f_y	MPa	7.5	460
Poisson's ratio, ν	---	0.3	0.3
Density, ρ	kg/m ³	1030	7850

Table 4. Material properties of soil mass

Parameters	Units	Value
------------	-------	-------

Density, ρ	kg/m ³	2100
Shear modulus, G	MPa	13.8
Poisson's ratio, ν	---	0.3
Cohesion, c	kPa	62
Friction angle, ϕ	°	26

Table 5. Parameters for interface simulation

Parameters	Unit	Value
Contact type	---	TIEBREAK
Friction coefficient for static	---	0.71
Friction coefficient for dynamic	---	0.56
τ_n	MPa	0.05
τ_s	MPa	1.15
D	mm	10

Table 6. Vertical acceleration of the proposed multi-layer pavement slab

Item	Field trial test	Numerical result	Deviation from field trial test
Max. vertical acceleration (m/s ²)	35,400	38,870	10 %

Table 7. Peak reading for total pressure cell

Item	Field blast test (kPa)	Numerical result (kPa)	Deviation from field trial test
TPC1	Destroyed	13,393	Sensor destroyed as pressure >> range
TPC2	273	267	2 %
TPC3	200	241	20 %

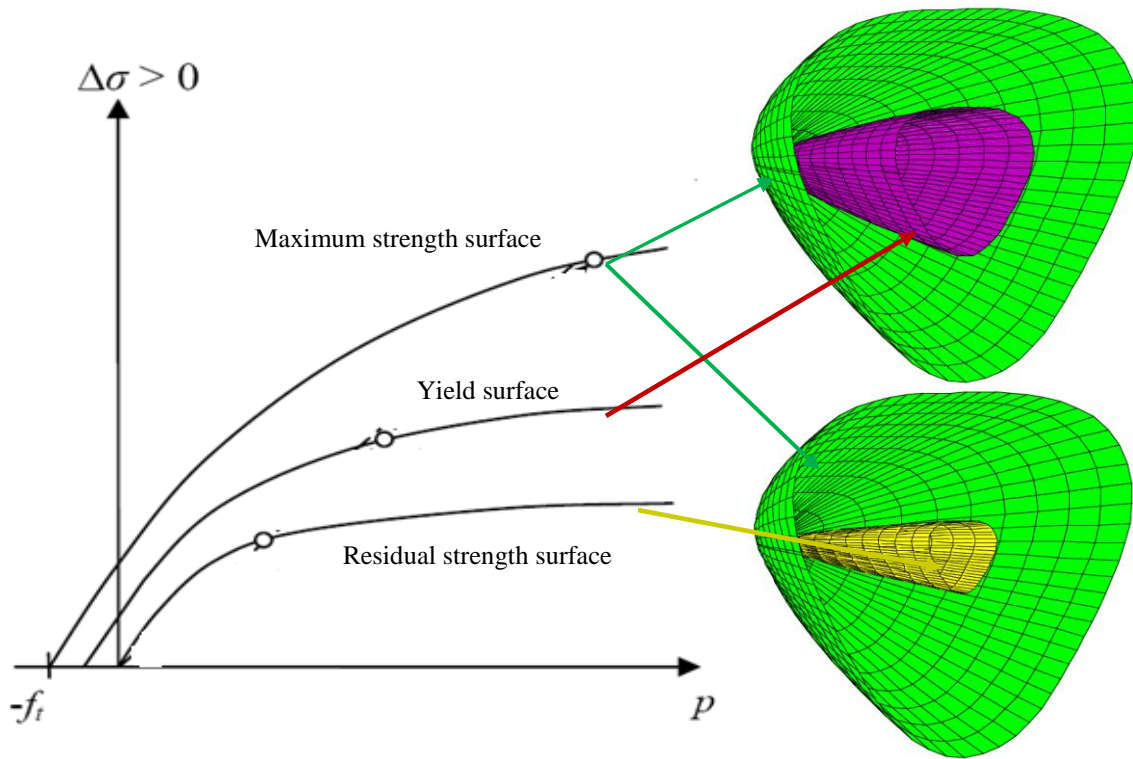


Figure 1. Strength surfaces for MAT72 R3 material model

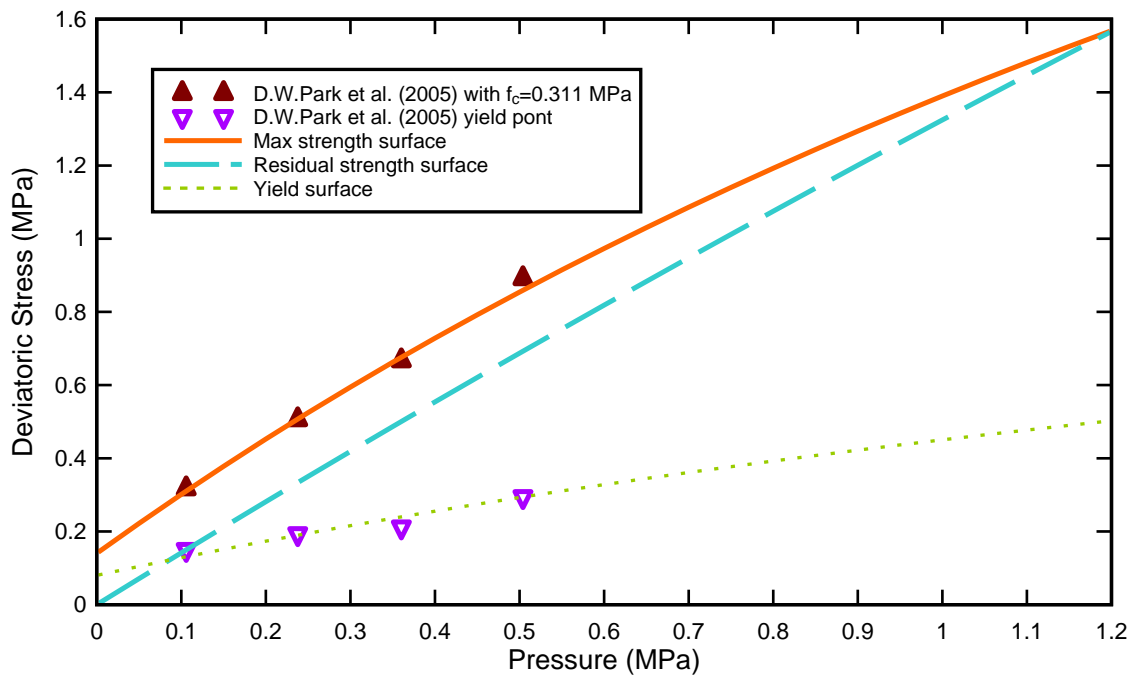


Figure 2. Determination of parameters in MAT 72 R3 from experimental data

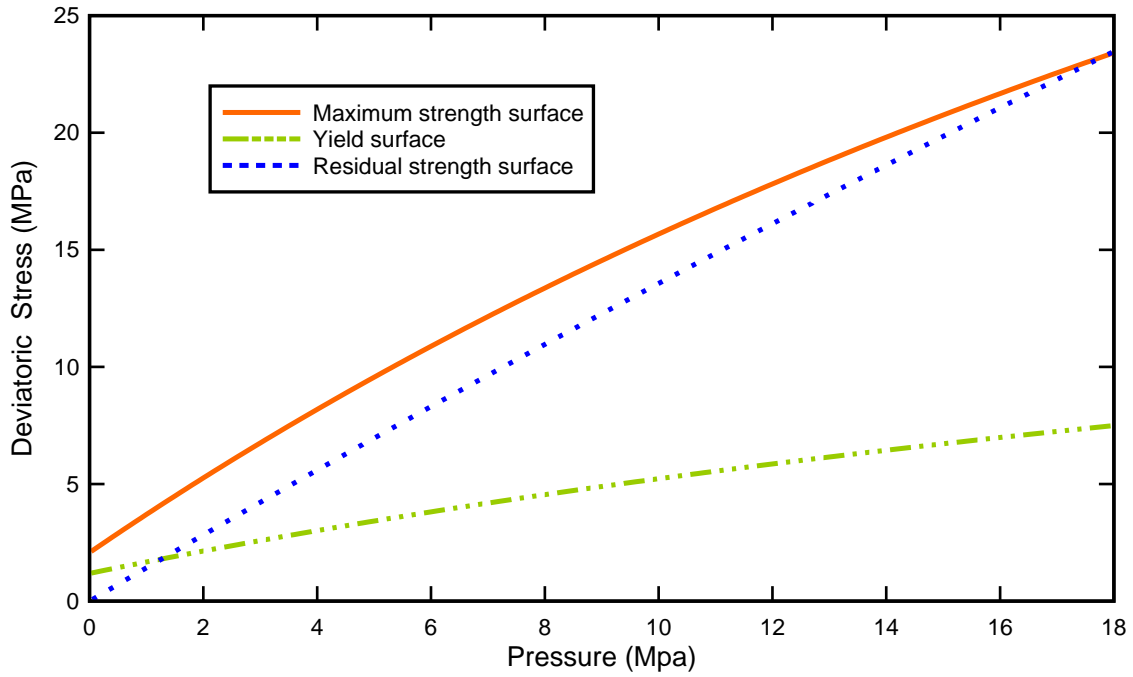


Figure 3. Strength surface for AC with $f_c=4.6$ MPa

Damage factor

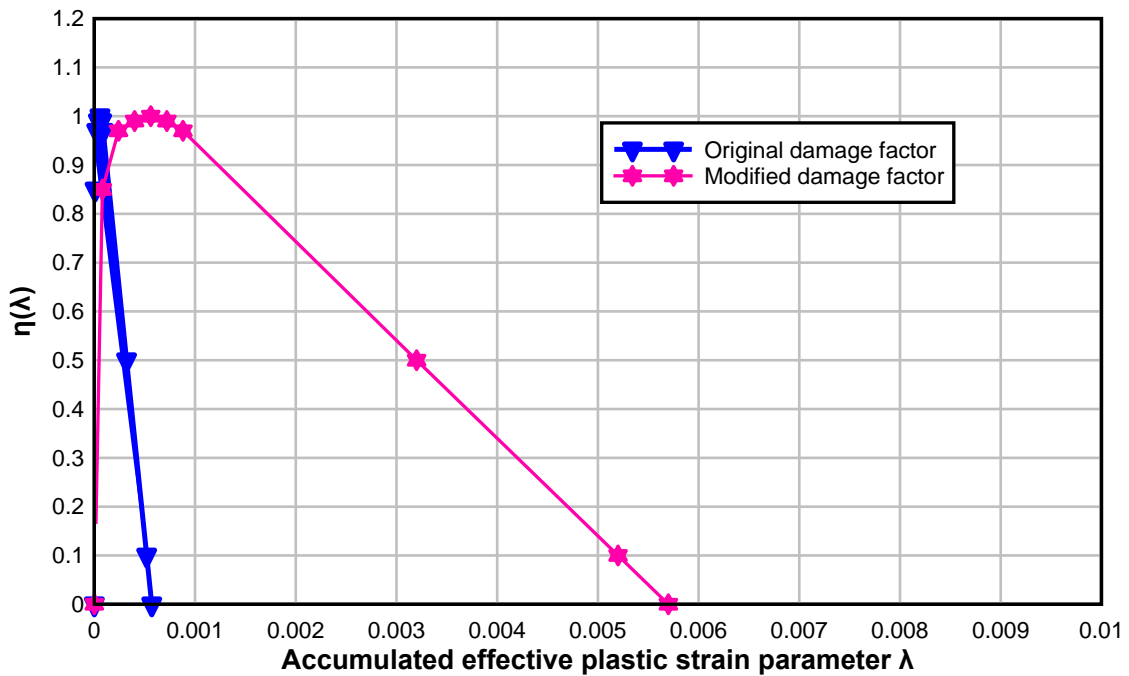


Figure 4. Damage factor used for AC material

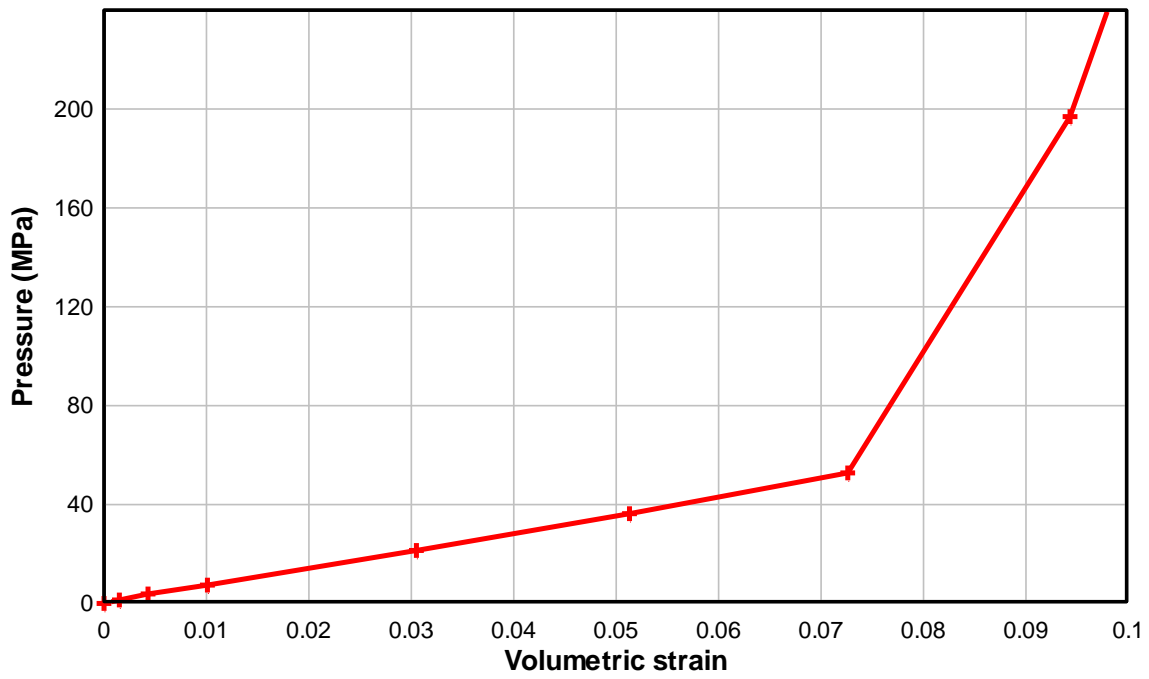


Figure 5. EOS for AC with $f_c=4.6$ MPa

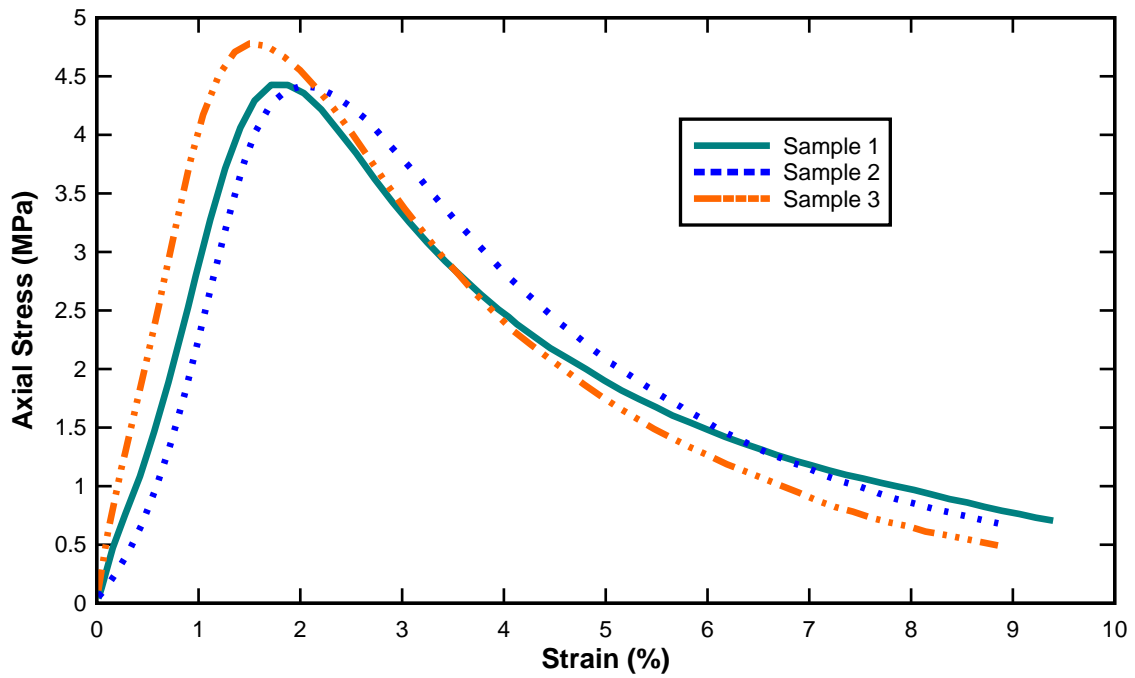


Figure 6. Stress-strain curve of uniaxial compressive test for AC

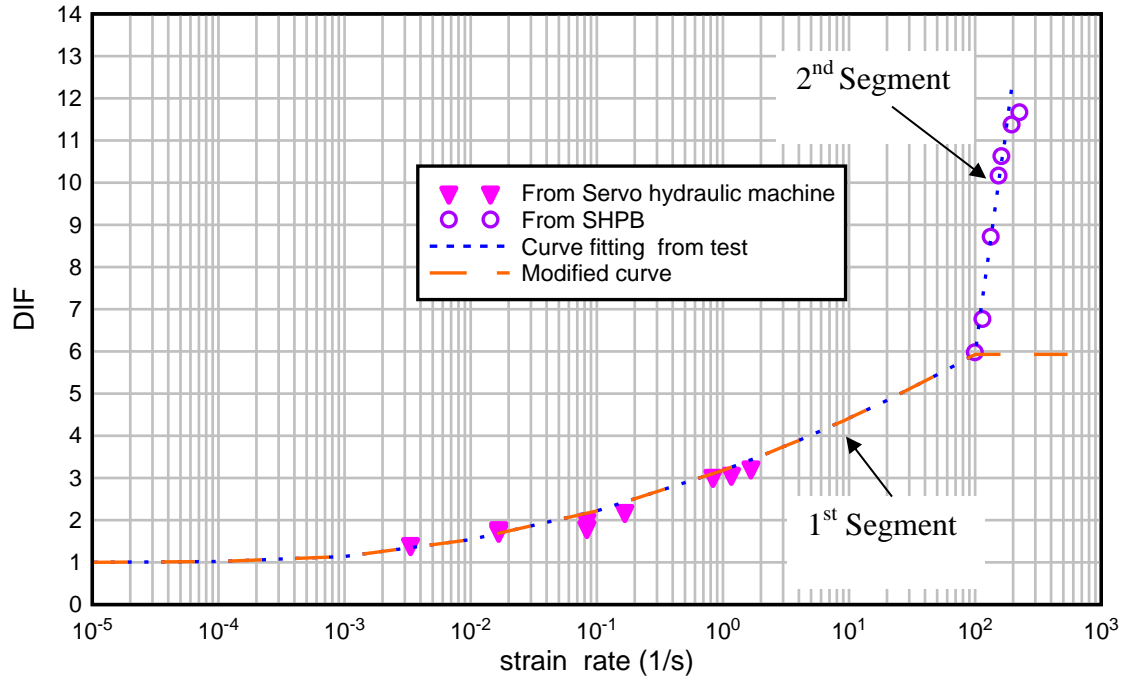


Figure 7. Compressive DIF curve versus different strain rate from lab test

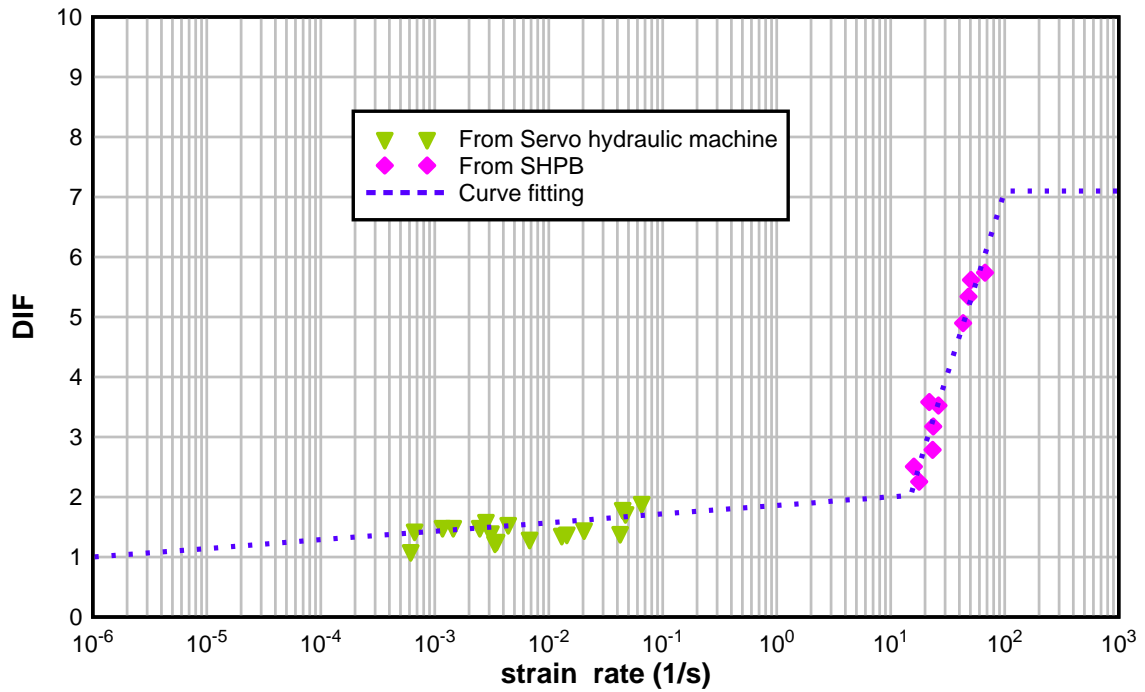


Figure 8. Tensile DIF curve versus different strain rate from lab test

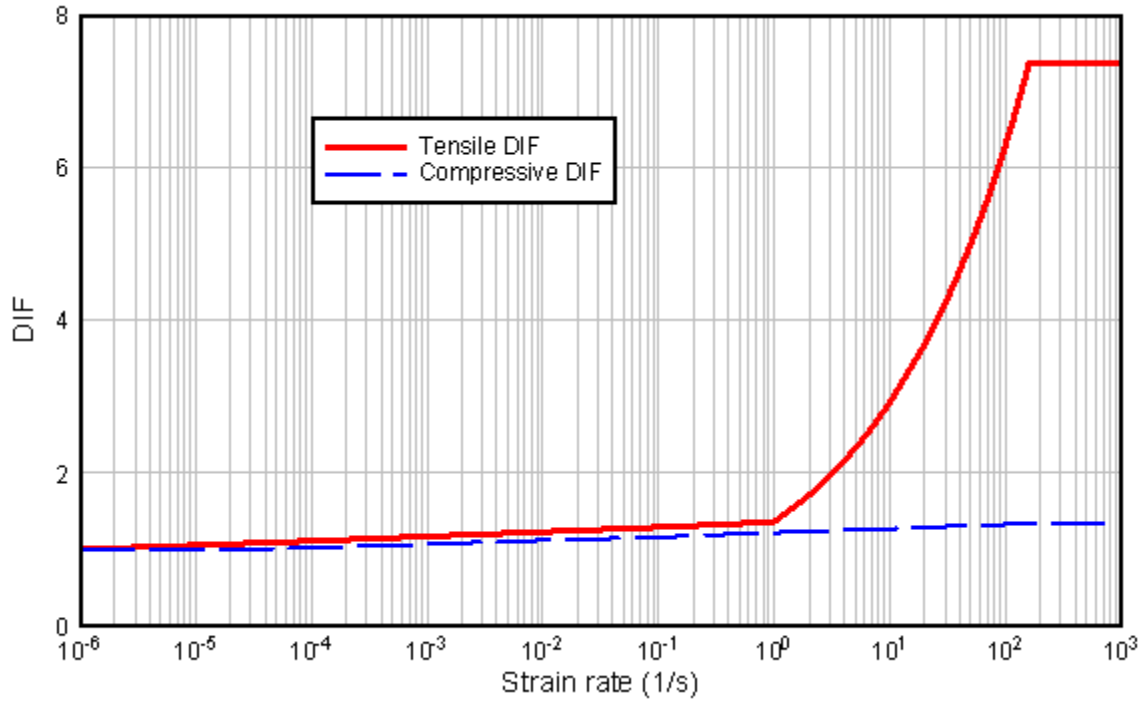


Figure 9. Tensile and compressive DIF curve used in numerical model for HSC with $f_c=55$ MPa

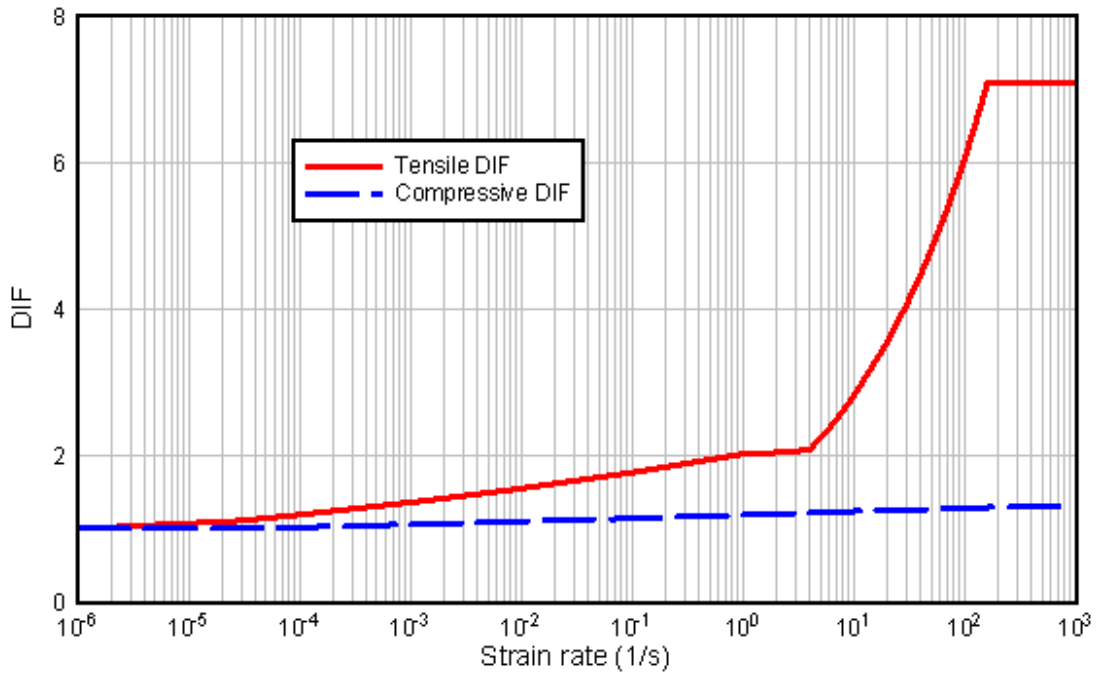


Figure 10. Tensile and compressive DIF curve used in numerical model for ECC with $f_c=64$ MPa

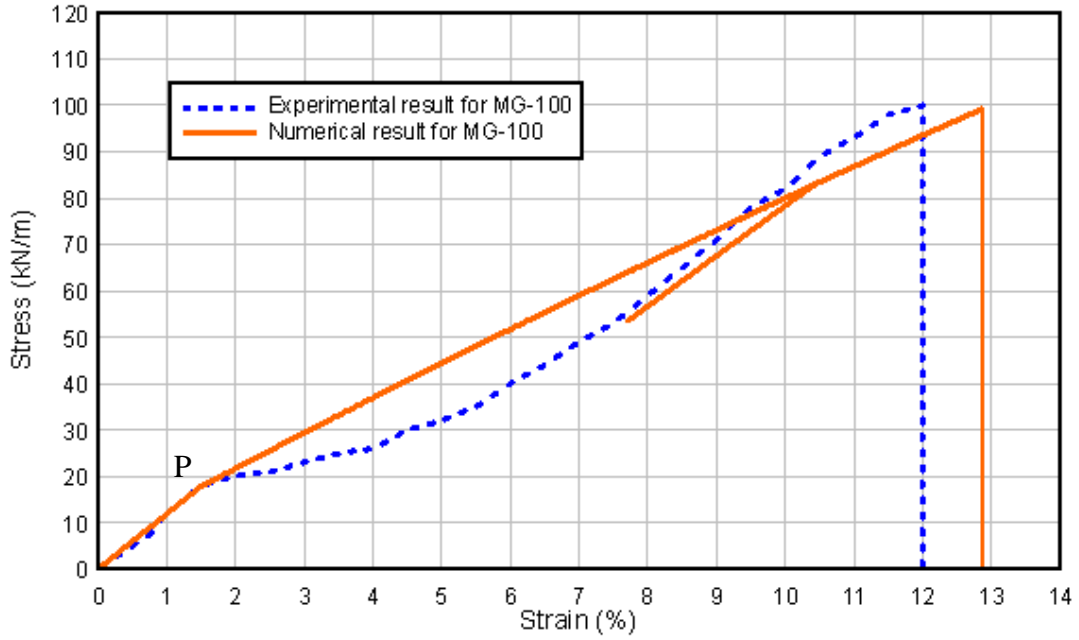
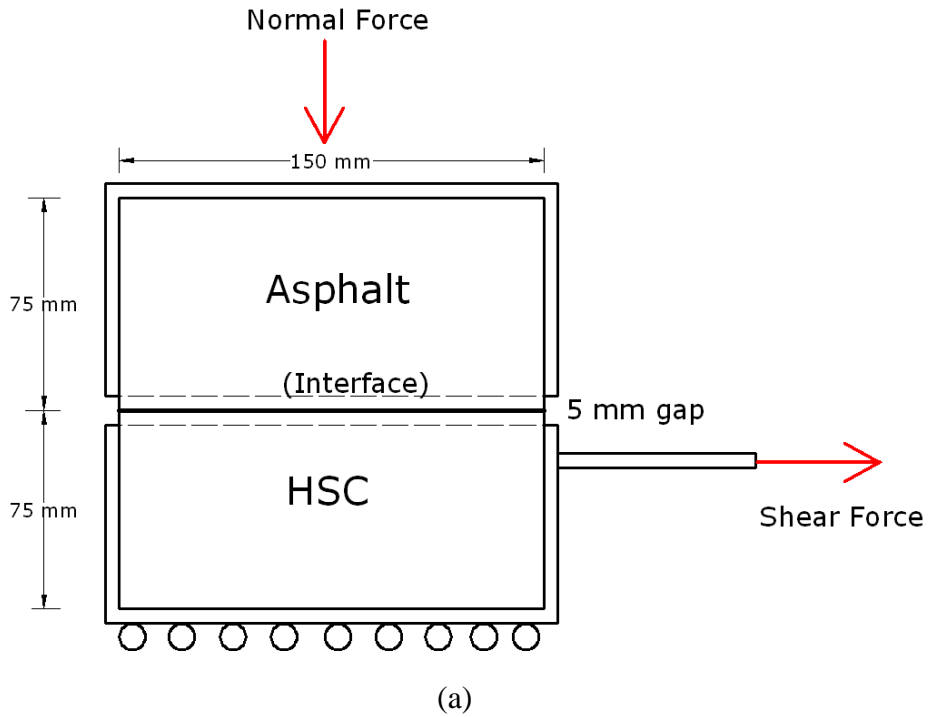


Figure 11. Load strain relationship of MG-100 GST reinforcements





(b)

Figure 12. Configuration of direct shear test, (a) Sketch of direct shear test, and (b) Apply normal force by hydraulic jack

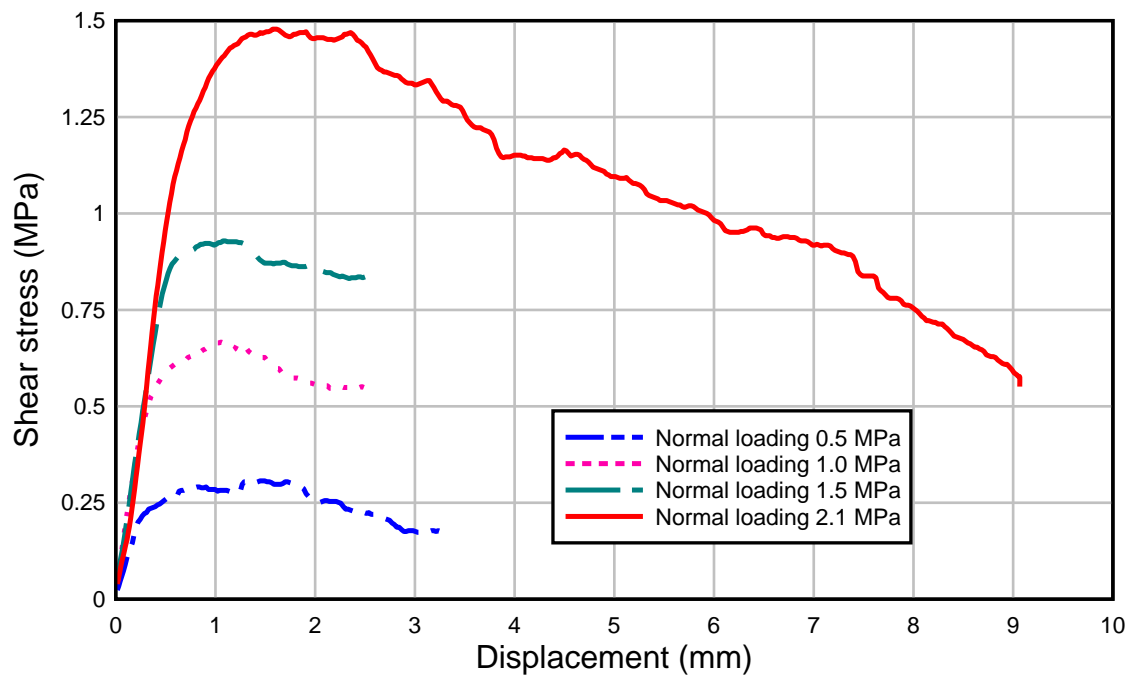


Figure 13. Shear stress and displacement in direct shear test

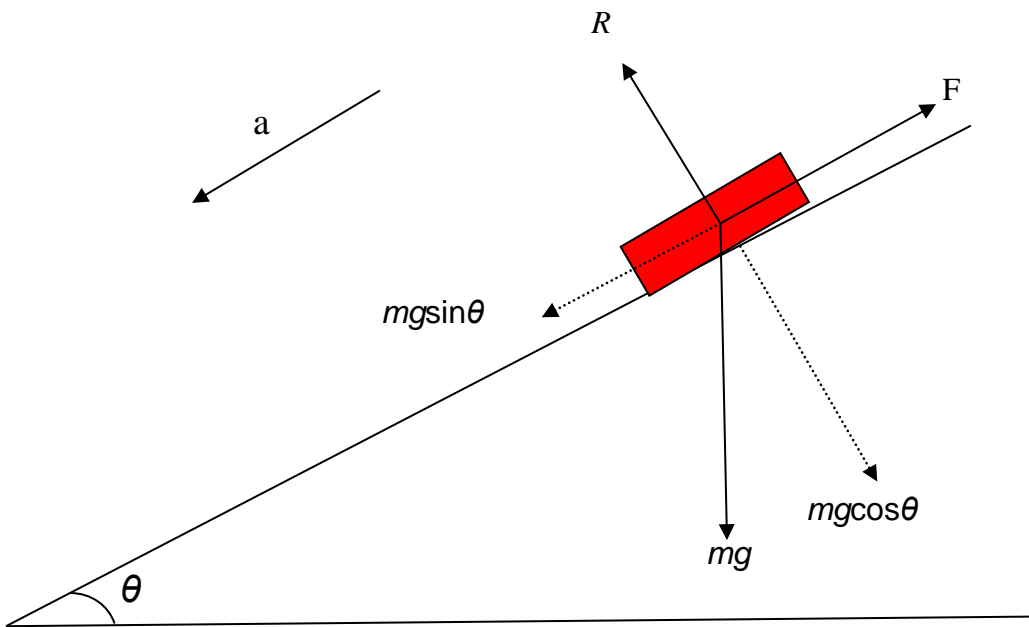
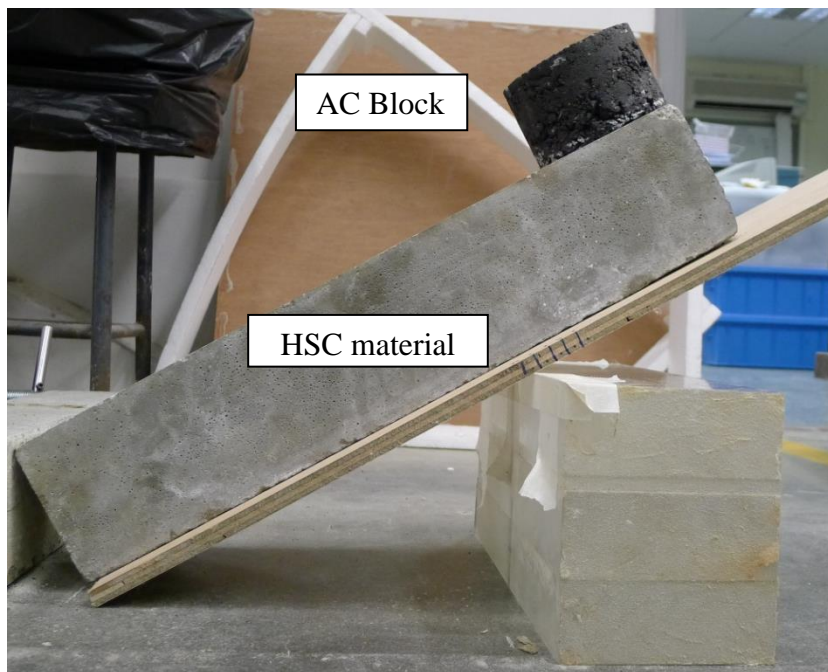
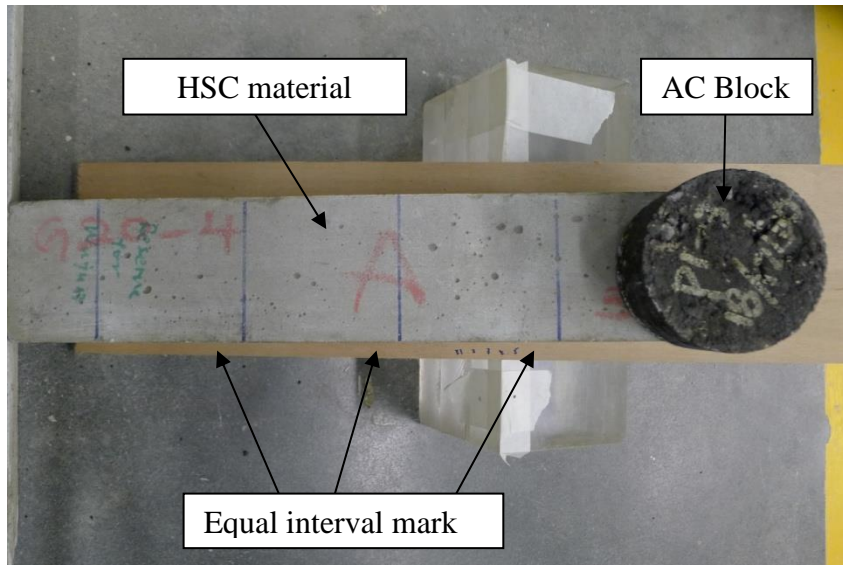


Figure 14. Tilt table test theoretical analysis



(a)



(b)

Figure 15. Set up for tilt table test: (a) Side view, and (b) Plan view

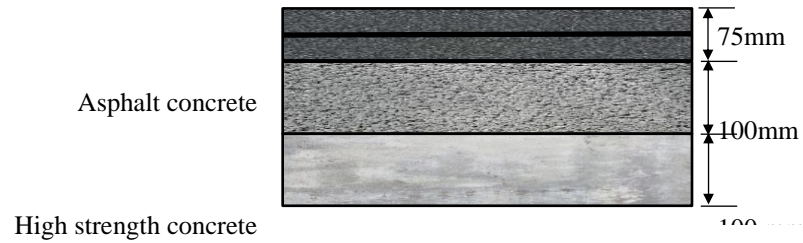


Figure 16. Configuration of the innovative SHS multi-layer pavement composite used in field blast test



Figure 17. Completed SHS multi-layer pavement slab with anchoring at site

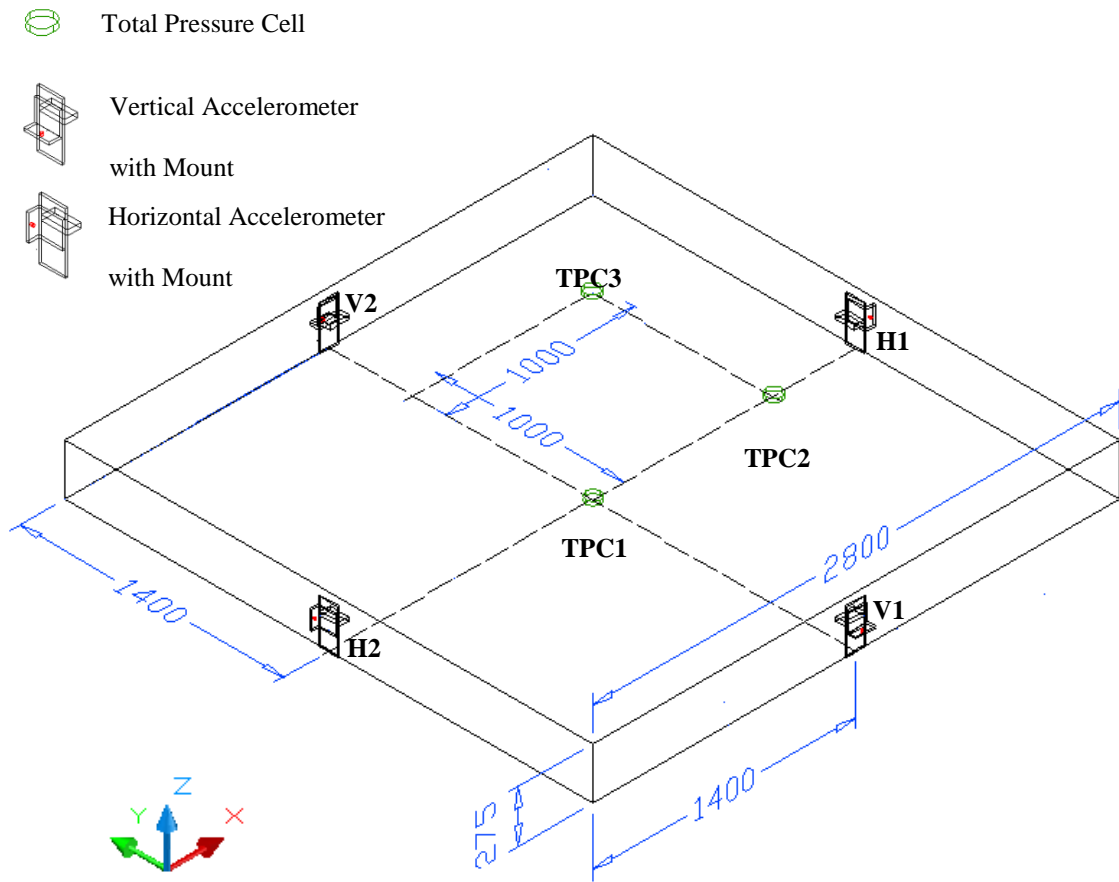


Figure 18. Layout of instrumentation for blast test

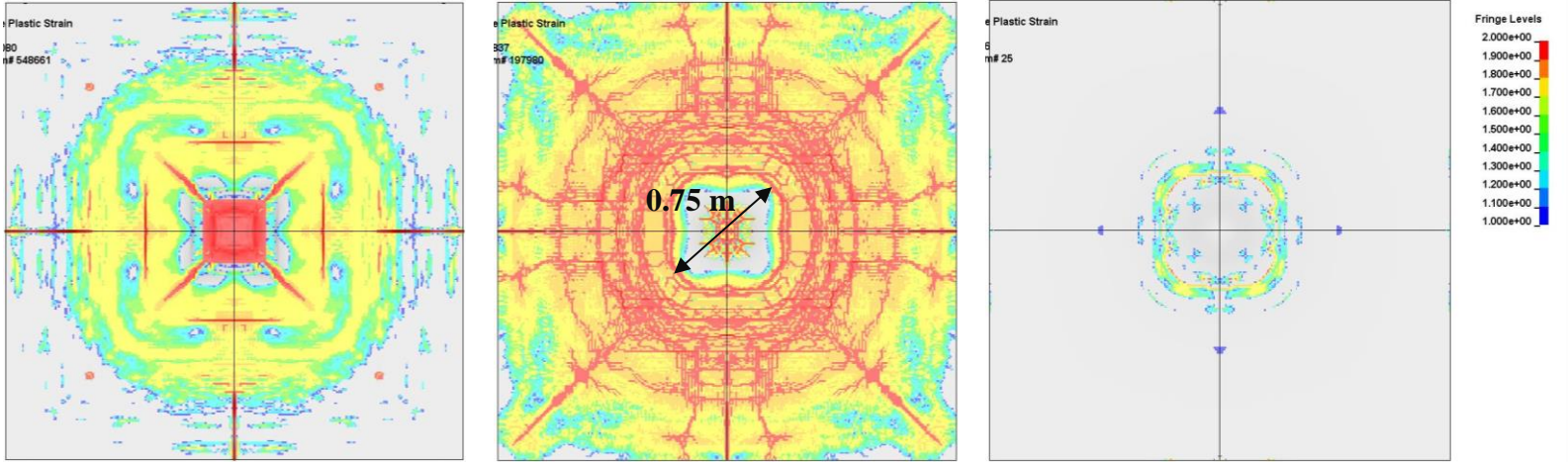


Figure 19. Damage of the multi-layer pavement after blast load with top layer AC in place

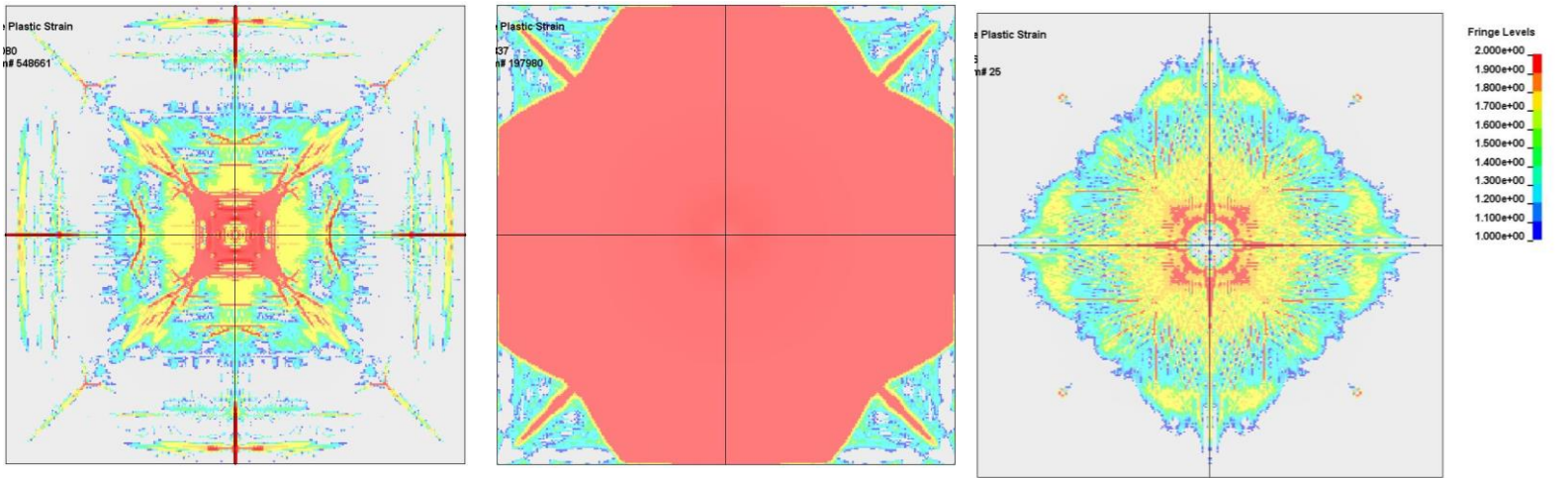


Figure 20. Damage of proposed multi-layer pavement after blast (after removing AC layer)

Top face



Bottom face



(a) AC layer

(b) HSC layer

(c) ECC layer

Figure 21. Damage pattern for each layer of proposed multi-layers pavement