A Mixture of Coal Wash and Fly Ash as a Pavement Substructure Material

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ABSTRACT

The reuse of waste materials in engineering projects has become the subject of many research efforts worldwide as it provides economical as well as environmental benefits. Coal wash (CW) and fly ash (FA) are example waste materials that can be used as alternative aggregates in transportation infrastructure projects, specifically as base and subbase materials in roads. Class C FA has been extensively used as a stabilizing material due to its hardening potential. However, Class F fly ash, a non-pozzolanic material when used alone, has not been considered in past research projects. In this study, Class F fly ash is mixed with coal wash as a void filler to enhance its compaction efficiency and produce a compact and well interlocked structure. A laboratory testing plan is performed to assess the geotechnical properties of the mixtures with 0%, 7%, 10% and 13% FA content and it includes compaction tests, unconfined compressive strength tests, California Bearing Ratio (CBR) tests, collapse potential tests and permeability tests. The mixture with 7% FA is selected as the optimum mixture and its potential for tensile cracking under service loads is further investigated using four-point bending tests. Also, the resilient modulus and permanent deformations of the mixture are evaluated under different dry-back conditions using multistage repeated load triaxial tests.

1 INTRODUCTION

The accumulation of waste materials has become a serious environmental and socio-economic concern in some parts of Australia where coal and steel industries are well established. Stockpiles of waste materials resulting from the operation of these industries, namely coal wash (CW) and fly ash (FA), occupy otherwise useable land surrounding the suburbs of regional areas. Moreover, the current shortage in good quality quarried aggregates in the proximity of major projects promotes the use of such granular wastes in engineering projects, and more specifically in transportation infrastructure projects like roads, railways and airfields.

The reuse of bituminous coal wash (e.g. Leventhal, 1996; Okagbue and Ochulor, 2007; Rujikiatkamjorn et al., 2013; Heitor et al. 2016), coal tailings (e.g. Indraratna et al., 1994; Williams et al., 1995; Morris and Williams, 1997; Williams and King, 2016) and mixtures of coal wash and other waste materials like steel furnace slag and rubber crumbs (Chiaro et al., 2014, Indraratna et al., 2018; Qi et al., 2018, Indraratna et al., 2019) was studied widely in the literature. Some of these studies have demonstrated that marginal materials like coal wash have acceptable geotechnical properties to be used as a construction fill in port reclamation projects (e.g. Rujikiatkamjorn et al., 2013; Chiaro et al., 2014, Tasalloti et al., 2015) or as a subballast layer in railways (Indraratna et al., 2018). However, the reuse of coal wash as base/subbase material in pavement construction has not been systematically explored in past studies on waste materials.

On the other hand, fly ash has been considered for a long time as a stabilizing agent in different applications such as base and subbase materials (Arulrajah et al. 2015; Gnanendran & Paul 2016; Paul et al. 2015; Sobhan & Das 2007; Sobhan & Mashnad 2002), construction materials (Ryu et al. 2013; Consoli et al. 2014,) and problematic soils (Kumar and Sharma 2004; Santos et al 2011; Horpibulsuk et al. 2012). Only few studies have assessed its properties as a construction material where fly ash was a main constituent of the of the mixture (Ghosh & Subbarao 1998; Kaniraj & Gayathri 2003; Kaniraj & Havanagi 2001; Kim et al. 2005; Pusadkar & Ramasamy 2005). Most of these studies considered Class C fly ash only due to its self-cementing properties. The use of coal fly ashes (class F) and other wastes (calcium carbide lime, reclaimed asphalt pavement, residues of NaOH, amongst others) has been common since the 1960's (e.g. Mateos 1961). The coal wash-fly ash compacted blends may suffer collapse due to water entrance. A way to avoid such problems would be stabilizing such residues with alkali activators that are also residues, such as carbide lime and/or residues of NaOH. However, Class F fly ash by itself is not self-cementing and hence has not been considered significantly in previous studies as a potential additive to other aggregates. The majority of coal combustion products (CCPs) produced in coal-fired power stations in Australia is categorized as Class F fly ash. The utilization of this waste material would help reduce the requirements for landfills and waste streams.

Depending on the source of coal wash, the material may not have enough fines to comply with the requirements for a base/subbase material. The absence of enough fines reduces the compacted density of the mixture, and consequently the mechanical interlock as well as the permeability. This makes coal wash not desirable to be used as road base/subbase (Austroads, 2008). In this study, FA is introduced into a CW mixture as a void filler to shift the particle size distribution (PSD) to be within the envelope specified for a base/subbase material and provide a fabric with reduced porosity and potential breakage of coal wash particles. Preliminary standard Proctor compaction tests on CWFA mixtures with 0% to 20% FA content were carried out to establish the mix with the highest Maximum Dry Density (MDD). In practice, field compaction energies are higher than standard Proctor compaction effort. Therefore, further compaction tests were performed at modified Proctor for CWFA mixtures with 7- 13% FA content to evaluate the potential use as road substructure (either base or subbase). The testing program consists of unconfined compressive strength tests, California bearing ratio (CBR) tests, collapse potential and permeability tests. The brittle behaviour of the proposed mixture and the potential for tensile cracking were assessed using four-point bending tests. Repeated load triaxial tests were also performed to explore the resilient modulus and the deformation properties of the mixture under dynamic loads.

2 MATERIALS AND TESTING PROGRAM

Coal wash (CW) was supplied by South32 and it was sourced from West Cliff colliery (New South Wales, Australia). Fly ash (FA) was procured from Eraring Power Station (New South Wales, Australia). The mineral components of CW and FA were determined using x-ray diffraction analysis, and the results are shown in Fig. 1. The mineral composition of the constituents is listed in Table 1. Fly ash used in this study can be classified as Class F because no calcium was detected. CW is a well graded material with particle sizes ranging between 0.075mm and 30 mm whereas fly ash consisted mainly of fine particles less than 300 μm. The PSD curves of the materials are shown in Fig. 2. The original PSD curve of CW does not meet the minimum percentage of fines (less than 75 μm) specified by Road and Maritime Services (RMS) for base/subbase material. However, for a FA content between 7% and 15%, the PSD curve of the mixture is within the lower and upper limit for a base/subbase material. Both CW and FA are less dense than traditional aggregates, having a specific gravity of 2.25 and 1.96, respectively.

Seven mixtures were considered for the preliminary compaction tests with FA content ranging between 0% and 20%. Based on the initial test, mixtures with narrow range of 7- 13% FA content only were considered for all subsequent testing. Fig. 3 shows a schematic diagram of the experimental plan followed. Coal wash material was sieved using both the wet and dry method as per the Australian Standards (Standards, 2009) and the material was separated into different grain size fractions. To reach the target PSD, each sample was prepared by obtaining the exact weight of each size and mixing them thoroughly with enough fly ash to reach the desired FA content. Finally, water was added and the sample was left to cure in a sealed container at constant temperature and humidity for 24 hours to ensure consistent water distribution in the mixture.

The compaction curves of the mixtures were studied at standard and modified Proctor efforts as per the Australian standards AS1289.5.2.1 and AS1289.5.1.1 (Standards Australia, 2017a&b). Samples were sieved after compaction to evaluate breakage using the change in the PSD curves before and after compaction, a concept introduced by Indraratna et al. (2005) for ballast as shown in Fig. 2. The CBR tests were carried out under soaked conditions as detailed in the Australian standard AS1289.6.1.1 (Standards, 2014). The samples were compacted to the desired dry density in five layers and then submerged in water for 4 days with a surcharge of 4.5 kg. Then the mould was removed from water and the penetration test was performed as detailed in the standard. Unconfined compressive strength tests were performed as per the Australian standard AS5101.4 (Standards, 2008). The tests were carried out at a constant speed of 1 mm/min until failure. Modified oedometer tests were performed to determine the collapse potential of the mixture. The samples were compacted at modified proctor effort and an axial load was applied progressively up to 200 kPa. Then the oedometer was flooded with water and left for 24 hours to saturate. The collapse potential was then determined as:

$$CP = \frac{\Delta e}{1 + e_0} \quad (1)$$

where Δe is the change in void ratio upon wetting and e_0 is the initial void ratio of the sample.

To evaluate the potential for tensile cracking of the compacted CWFA mixture, the tensile strength of the mixture was examined using four-point bending tests with a constant maximum bending moment at the centre of the beam. Five specimens were compacted in a 100 mm x 100 mm x 345 mm mould using modified Proctor effort at different moisture contents on the wet and dry sides of OMC (Table 2). The beams were then loaded at constant rate of 1 mm/min and the load and corresponding deformations were recorded automatically. The tensile strength was then calculated as:

$$\sigma_f = \frac{FL}{bd^2} \quad (2)$$

where F is the maximum load applied and L, b, and d are the length, width and depth of the beam, respectively. The effect of water content on the tensile cracking was assessed by plotting the variation of the tensile strain ratio with respect to water content. The tensile strain ratio is calculated as:

$$Tensile\ strain\ ratio = \frac{\varepsilon_t - \varepsilon_{t_{optimum}}}{\varepsilon_{t_{optimum}}} \quad (3)$$

where ε_t is the tensile strain at a given moisture content and $\varepsilon_{t_{optimum}}$ is the tensile strain at the optimum moisture content. A similar approach was adopted by Indraratna et al. (1990) to explore the cracking potential of a compacted lateritic residual soil.

Based on preliminary test results, specimens with 100 mm diameter and 200 mm height were prepared at 7% FA and compacted at modified proctor effort for repeated load triaxial tests. In practice, a base/subbase layer is compacted at optimum moisture content (OMC), but then undergoes dry back before the overlying layer/seal is placed. The mixtures were compacted at OMC and then left to dry until the target water content (70-90% OMC). The multi-stage cyclic loading was applied in accordance with the Austroads method (2007a) at a confining pressure of 50 kPa (Table 3).

3 EXPERIMENTAL RESULTS

3.1 Compaction Characteristics and Breakage Evaluation

The compaction characteristic curves at standard Proctor energy of the mixtures (0-20% FA) are shown in Fig. 4(a). At standard Proctor compaction, the maximum dry density (MDD) increases with increasing FA content up to 10%, after which the MDD of the mixture decreases again. To eliminate the role of specific gravity, and evaluate the effect of FA on the pore volume within the compacted mixture, compaction data is represented in terms of void ratio in Fig. 4(b). The void ratio decreases from 0.35 when no FA was added to 0.26 at 10% FA content which corresponds to 25% decrease in void space. This indicates that FA is acting as a void filler and thus reducing the pore volume within the fabric. It can be seen that CWFA with 7-13% can provide relatively higher compacted density.

In the field, it is observed that compaction energy is often higher than the laboratory standard compaction. Additional modified Proctor compaction tests were performed on a narrower range of FA content (i.e. 7%, 10% and 13%) based on the results from standard Proctor compaction (Fig. 4(c)).

The MDD reaches a maximum at 7% FA and decreases again when FA content is at 10% and 13%. When more FA is added, it is likely that the fine particles occupy space that could otherwise be occupied by larger and denser CW particles, thus reducing the MDD of the mixture. The optimum moisture content (OMC) increases with increasing FA content from 7% to 13%, which could be attributed to an increase in the specific surface of particles when the fines content increases. It can be seen that the compaction curves are relatively flat and a relative compaction greater than 98% could be achieved within a water content range between 4% and 8%. However, the significant drop in compaction efficiency on the wet side of OMC is more evident than that on the dry side. Also, void ratio contour lines in Fig. 4(d) clearly show that the minimum pore volume is at 7% FA and 6% water content (OMC), which is in agreement with the compaction curves shown in Fig. 4(c). However, while the compaction curves show that at 13% FA the MDD is lower than that at 100% CW, the void ratio at 13% FA is lower than that at 100% CW, which further proves that the void ratio is a better indicator of the compaction efficiency at different FA contents. The experimental results at modified Proctor are compared with compaction data retrieved from two sites where the CWFA mixture was used. At these sites, the CWFA material had FA content at 7% similar to the mixture. It is clear that modified Proctor energy is more appropriate to simulate field conditions as the experimental data falls within the filed compaction range.

The breakage index (BI) of the mixtures after modified Proctor compaction are listed in Table 4. FA does not have a significant effect on breakage up to 10% as it is mainly acting as a void filler. At 13%, breakage starts to decrease because CW particles are replaced with FA fines and hence a smaller portion of the total mixture is prone to breakage.

3.2 Geotechnical Properties of the Compacted Mixture

3.2.1 Unconfined Compressive Strength

The unconfined compressive strength of the mixtures with 7%, 10% and 13% FA compacted at modified Proctor effort was explored at four different moisture contents relative to OMC and the results are shown in Fig. 5(a-d). The samples were sheared immediately after compaction because no hardening reactions were expected. The unconfined compression test (UCS) of base materials is required to be less than 1000 kPa to prevent highly brittle behaviour (RMS, 2014). While all the mixtures comply with the above specification, at OMC the highest UCS of 240 kPa is observed at 7% FA content. As expected, the maximum axial stress decreases with increasing water content on the wet side of OMC for all mixtures and a ductile behaviour is dominant, i.e., a decrease in stiffness is clearly observed (Fig. 5e). All mixtures show a significant post-peak brittleness on the dry side of OMC. At water content less than optimum moisture content by 2% (OMC-2), the peak axial stress remains relatively the same for 100% CW, decreases for 7% and 10% FA and increases for 13% FA. It requires further decrease in water content below than optimum moisture content by 4% (OMC-4) for the peak axial stress to increase for a mixture with 10% FA. This suggests that for the tested material, compaction should be performed very close to OMC to mobilize the maximum strength that the material can sustain without inducing a significant post-peak brittle failure and wet conditions should be avoided because they lead to a substantial drop in strength. Figure 5f plots the effect of water content on the axial strain at failure. The same trend is observed on the wet side of the OMC for all the mixtures where the axial strain at failure increases with increasing water content, i.e. the stress-strain relationship shifts towards a ductile behaviour. The change in the axial deformation on the dry side of OMC is not as evident as the wet side suggesting that the performance of the material is very sensitive to wet conditions which should be avoided in practice. At OMC, the minimum axial strain at failure was at 7% FA. Therefore, from a practical point of view, the UCS test results indicate that the mixture with 7% FA can sustain the highest axial load with the minimum corresponding axial deformation.

3.2.2 California Bearing Ratio (CBR)

The strength of the CWFA mixtures as a potential base/subbase material was further explored using soaked California Bearing Ratio tests (CBR). The results for all the mixtures considered in this study are shown in Fig. 6. All the mixtures were compacted at the OMC under modified Proctor compaction effort and then submerged in water for four days. At 7% FA content, the CBR increases from 72% to 125%, which is comparable to well graded clean gravels used in the construction of roads (Indraratna, 1994). The CBR values decrease again when FA content increases beyond 7% and drops to 66% and 58% at 10% and 13% FA content, respectively. VicRoads standards for crushed rock materials (VicRoads, 2017) specify a minimum CBR of 100 for Class 2 materials, a high quality base material for unbound flexible pavements, and 80 for Class 3 materials, a high quality upper subbase material for heavy duty unbound flexible pavements. Only at 7% FA, the CBR complies with the specific requirement for base and subbase materials. It is noted that this FA content corresponds to the lowest void ratio can be achieved at modified Proctor compaction, the highest UCS and the lowest axial deformation at failure. Hence, two main advantages of adding FA can be inferred; First, introducing an optimum amount of fines into the mixture improves the compaction efficiency and yields a more compact fabric with minimum pore volume. Second, reducing the pore volume within the compacted mixture reduces settlements and CW breakage when the mixture is subjected to service loads and improves the bearing capacity and longevity of the material.

3.2.3 Collapse Potential and Permeability

The collapse potential is an important parameter to investigate the potential deformation of the material when subjected to sudden flooding. To explore the collapse potential of the proposed mixture, oedometer tests were performed on CWFA mixtures with 0%, 7%, 10% and 13% under modified proctor compaction and the samples were flooded at an axial stress of 200 kPa. Permeability was also measured after compaction and after inundation to evaluate the effect of change in void ratio and water content on the hydraulic properties of the material. Figure 7 shows that the collapse potential of all the mixtures is well below the acceptable value of 1% (Jennings and Knight, 1975). Nevertheless, CP decreases by 70% when 7% FA is added, after which no significant change is observed. Figure 7 also shows that permeability decreases with increasing FA content. This indicates that FA acts as a void filler and reduce the pore space within the mixture. Despite the decrease in void ratio after flooding, the permeability of the all the mixtures increases slightly. This could be attributed to the fact that the specimens were unsaturated right after compaction. When inundated, the water content increases and the specimens approach a saturated state at which permeability is slightly higher. However, permeability remains less than $5x10^{-7}$ m/sec for all CWFA mixtures even after inundation and the material can still be considered impermeable for base/subbase applications.

3.2.4 Tensile strength

Based on compaction, UCS, CBR and collapse potential tests, a 7% FA content provides a mixture with maximum compaction efficiency, highest strength, minimum axial deformation and acceptable settlement upon wetting. To further evaluate the potential use of the mixture as a base/subbase, the potential for tensile cracking was studied using four-point bending tests at different water contents relative to OMC. The stress-strain curves and the variation of the tensile strain ratio with water content are shown in Fig. 8. As expected, brittleness of the material is more evident on the dry side of OMC. When compacted at OMC, the mixture exhibits the highest tensile strength. At water content below 1% OMC (OMC-1), the mixture maintains a relatively high tensile strength with a tensile strain ratio close to zero. However, the tensile strength drops significantly when the water content decreases further (OMC-2) and the tensile strain ratio reduces 50%. On the other hand, a ductile behaviour was observed on the wet side of OMC with a tensile strain ratio increases 200% at OMC+2. Nevertheless, the increase in ductility is accompanied with a 50% reduction in strength. This indicates that, when the mixture is compacted on the wet side of OMC, tensile cracks would develop under lower loads than when the mixture is compacted at OMC or OMC-1. Also, very dry conditions should be avoided because tensile cracks would occur under very low axial loads.

3.3 Resilient Modulus and Permanent Deformations under Dynamic Loading

The resilient modulus is a mechanical property widely used in pavement design to describe the stress-strain response of a material under cyclic loading. The resilient modulus is calculated as the ratio of the dynamic stress and the associated strain and is determined using the repeated load triaxial test (RLT), the dynamic loading test specified by Austroads pavement guide (Austroads, 2008) for base and subbase materials. The test was carried out in five stages starting with a cyclic deviator stress of 150 kPa and then increased the stress by 100 kPa at each subsequent stage. The resilient modulus of the mixture with 7% FA was evaluated at OMC and three dry back conditions (i.e. 90% OMC, 80% OMC and 70% OMC) to explore the sensitivity of the material's resistance to deformations to variations in water content after placement (Fig. 9). The permanent deformations incurred at different dry back conditions and loading conditions are also plotted.

The specimen tested at OMC shows a notable increase in permanent strain with the number of cycles at the beginning of stage 3 and then experiences a frictional failure at the end of stage 4 indicating a possible bearing capacity failure at a stress level greater than 350 kPa. The permanent strain decreases with dry back up to 80% OMC to less than 1% but then increases again when the specimen was tested at 70% OMC. Nevertheless the strain remains relatively constant with increasing stresses at that water content, i.e. when the mixture is densified through the first thousand cycles, it becomes more resistant to deformations than other mixtures tested at higher water contents. At 90% OMC, the mixture starts approaching a shear failure at the beginning of stage 5 with a notable increase in permanent strain. In terms of expected permanent settlements under service loads, the permanent strain of the mixture remains below the failure limit suggested by Austroads (2007b) for base/subbase aggregates for all dry back conditions and only exceeded that limit at OMC at the beginning of stage 5 at a stress level equivalent to 40 kN single tyre load.

The resilient modulus of the mixture increases slightly with increasing applied stress because the material experiences densification. The resilient modulus is expected to increase with the reduction in water content (i.e. dry back) partly due to the suction developed in the pore space within the mixture. During stage 1, the resilient modulus increases from 63 MPa at OMC to 96 MPa at 80% OMC and then decreases slightly at 70% OMC. However, when the stress is increased at stage 2, the specimen at 70% dry back has the highest resilient modulus. Again, this is explained by the fact that the mixture went through a densification during the first cyclic loading stage. Sliding and rearrangement of particles occurred and the void ratio decreased further, thus increasing the stiffness of the mixture.

4 CONCLUSIONS

A laboratory investigation was conducted to evaluate the potential reuse of a coal wash and fly ash mixture with 0%, 7%, 10% and 13% FA content as base/subbase material in pavements where FA is primarily used a avoid filler. Initial testing included compaction, unconfined compressive strength, California bearing ratio and collapse potential and the following conclusions are made:

- Adding 7% FA is enough to increase to amount of fines in the mixture and yield a minimum void ratio at modified Proctor compaction. At higher FA contents, the compaction efficiency starts diminishing, thus reducing the interlock between particles.
- The mixture with 7% FA has the highest UCS and lowest axial deformation when compacted at OMC. The strength drops considerably when the mixture is compacted wet of OMC but maintains a relatively higher strength on the dry side of OMC. A CBR greater than 100, the minimum value required for base material by VicRoads, is only achievable at 7% FA.
- The collapse potential of all CWFA mixtures tested in this study is below the acceptable range. Therefore, no sudden and considerable settlements are expected to occur if the material is wetted. Also, all the mixtures are resistant to moisture penetration with a permeability less than $5x10^{-7}$ m/sec.

Further tests were conducted on the mixture with 7% FA to evaluate its properties as a base/subbase material. The following conclusions can be made:

- The potential for tensile cracking is very sensitive to moisture variations and is minimum when the mixture is compacted at OMC and OMC-1. Very dry conditions and wet conditions should be avoided as the tensile strength drops considerably and the potential for tensile cracking under service loads increases.

- The resilient modulus and permanent strains under cyclic loading are moisture-dependent and stress-dependent. The mixture failed under the stress levels of a base layer when tested at OMC and 90% OMC. It can only be used a base material if a dry back condition of at least 80% is achieved or in light traffic roads where the load is not expected to exceed 350 kPa. The mixture can be used with minimum settlements (< 1% strain) as a subbase, but 90% dry back conditions are preferred to avoid shear failure under very high traffic loads. Also, dry back conditions less than 70%OMC (i.e. OMC-1.8) should be avoided because it could lead to tensile cracking. The strength and stiffness of the material can be further improved by adding lime or cement to activate the pozzolanic reaction within the mixture but brittleness of the material should be carefully evaluated.

Although the inclusion of a detailed life cycle costing will be beneficial to the readership, this evaluation is not within the scope of this particular technical paper, and this is because the costs can vary significantly based on numerous factors including the source and type of coal wash, geographic location of sites, hence transportation issues, economies of scale, etc. In a past study, conducted by the 3rd Author as form of technical consulting in the past, a benefit to cost ratio of about 8-10 was evaluated for coal wash coming from the Wollongong region based mainly on material and transport costs, and that is all we can mention at this stage in this paper.

ACKNOWLEDGMENTS

Financial assistance provided by the Australian Research Council (ARC) Linkage Project (LP160100280) and industry partners Road and Maritime Service, Douglas Partners and South32 is gratefully acknowledged. Also, the assistance provided by Mr. Duncan Best and Mr. Richard Berndt in the performance of the laboratory tests is appreciated. The authors would also like to acknowledge the assistance provided by South 32 and Eraring Power Station in the procurement of the CW and FA material used in this study.

NOTATIONS

$\boldsymbol{CP}$	collapse potential
$\boldsymbol{CBR}$	California bearing ratio
$\boldsymbol{UCS}$	unconfined compressive strength
$\boldsymbol{\Delta e}$	change in void ratio at inundation
$\boldsymbol{e_0}$	initial void ratio
$\boldsymbol{\varepsilon_t}$	tensile strain
$\boldsymbol{\varepsilon_{t_{optimum}}}$	tensile strain at OMC
$\boldsymbol{\sigma_f}$	tensile/flexural stress
$\boldsymbol{F}$	maximum axial load applied
$\boldsymbol{L}$	length of the beam
$\boldsymbol{b}$	width of the beam
$\boldsymbol{d}$	depth of the beam

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Table 1: Mineral components of coal wash and fly ash by weight

Minerals	Composition (%)	
Quartz	35.5	38.5
Kaolinite	24.8	2.6
Mixed layer illite/smectite	14.1	
Illite	12.5	5.9
Siderite	6.9	
Labradorite	2.2	
Orthoclase	2.2	
Anorthite	1.5	
Calcite	0.3	
Mullite (Al_2O_3:SiO_2 1.71)	-	53.0

Table 2: Details of the experimental testing program

Compaction Tests			California Bearing Ratio tests		
Test number	**FA content (%)**	**Compaction effort**	**Test number**	**FA content (%)**	**Moisture content**
SP1	0	Standard Proctor ($596\ kJ/m^3$)	**CBR1**	7	OMC
SP2	5				
SP3	10				
SP4	15		**CBR2**	10	OMC
SP5	20				
MP1	7	Modified Proctor ($2703\ kJ/m^3$)	**CBR3**	13	OMC
MP2	10				
MP3	13				

Unconfined Compressive Strength tests					
Test number	**FA content (%)**	**Water content relative to OMC**	**Test number**	**FA content (%)**	**Water content relative to OMC**
UCS1	0	OMC-2	**UCS9**	10	OMC-4
UCS2	0	OMC	**UCS10**	10	OMC-2
UCS3	0	OMC+2	**UCS11**	10	OMC
UCS4	0	OMC+4	**UCS12**	10	OMC+2
UCS5	7	OMC-2	**UCS13**	13	OMC-4
UCS6	7	OMC	**UCS14**	13	OMC-2
UCS7	7	OMC+2	**UCS15**	13	OMC
UCS8	7	OMC+4	**UCS16**	13	OMC+2

Collapse potential tests		
Test number	**FA content (%)**	**Applied stress at inundation**
CP1	0	200 kPa
CP2	7	200 kPa
CP3	10	200 kPa
CP4	13	200 kPa

Four point bending tests						
Test number	**FA content (%)**	**Moisture content (%)**	**Beam dimensions (mm)**	**Compaction effort**	**Support span (mm)**	**Loading span (mm)**
B1	7	OMC - 2 (4%)	100x100x345	Modified Proctor ($2703\ kJ/m^3$)	260	85.3
B2		OMC - 1 (5%)				
B3		OMC (6%)				
B4		OMC + 1 (7%)				
B5		OMC + 2 (8%)				

Table 3: Details of the repeated load triaxial tests

Test number	**C1**		**C2**		**C3**		**C4**	
Dry back condition	**OMC**		**90% OMC**		**80% OMC**		**70% OMC**	
Multi-stage cyclic testing	Stage number	q_{cyc} **(kPa)**	Stage number	q_{cyc} **(kPa)**	Stage number	q_{cyc} **(kPa)**	Stage number	q_{cyc} **(kPa)**
	1	150	1	150	1	150	1	150
	2	250	2	250	2	250	2	250
	3	350	3	350	3	350	3	350
	4	450	4	450	4	450	4	450
	5	550	5	550	5	550	5	550

Table 4: Summary of test results

FA Content (%)	0	7	10	13
MDD (t/m^3)	1.77	1.81	1.78	1.75
Void ratio	0.27	0.23	0.25	0.26
OMC (%)	6.5	6.1	9.0	9.2
Breakage Index, BI (%)	25[a]	23	24	21
Fines content after compaction (%)	-	14.5	16.3	18.5
CBR (%)	72[b]	125	66	58
UCS at OMC (kPa)	200	240	207	187
Collapse potential (%)	0.35	0.1	0.1	0.05

[a] Rujikiatkamjorn et al. (2013)
[b] Indraratna (1994)
MDD-Maximum dry density, OMC- optimum moisture content, UCS- Unconfined compressive strength, CBR – California bearing ratio.

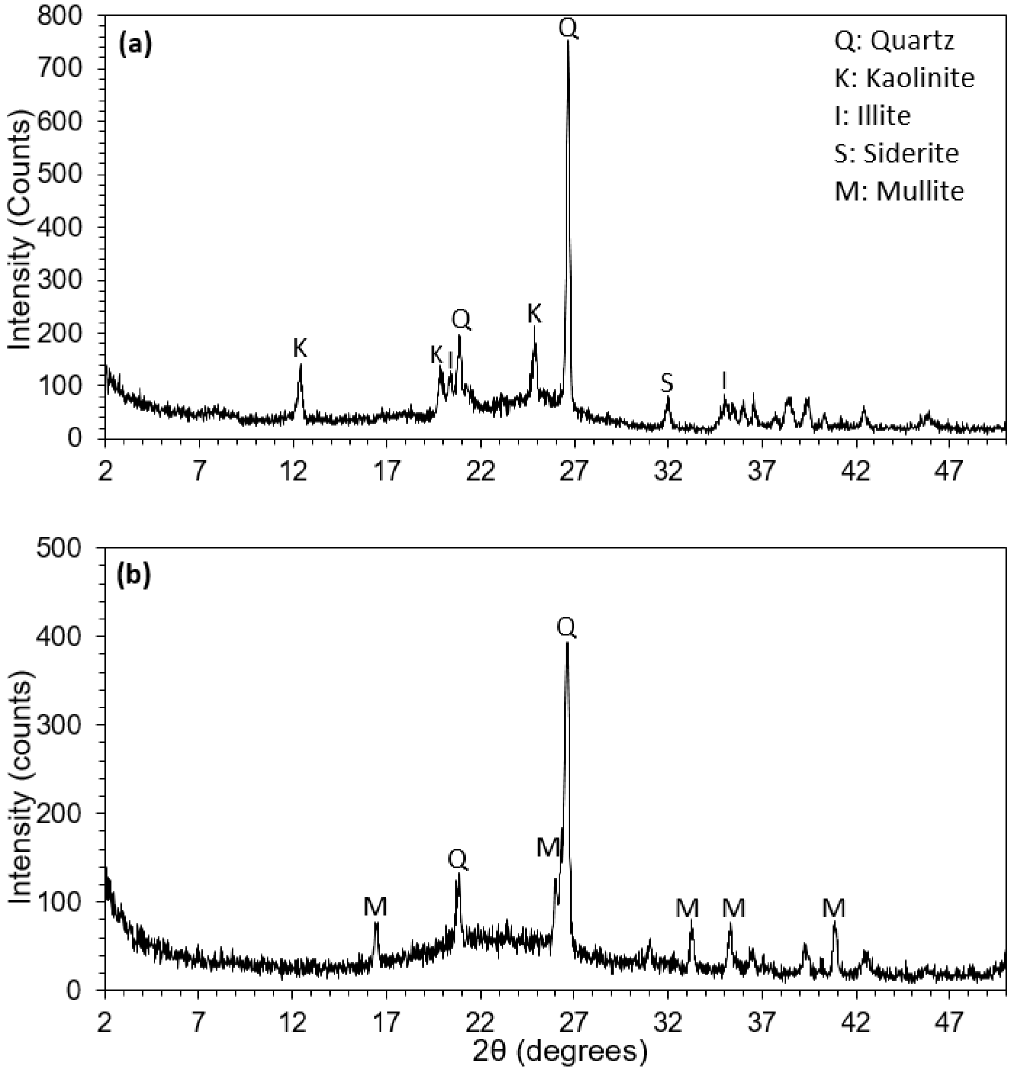

Figure 1: X-Ray diffraction of (a) coal wash and (b) fly ash

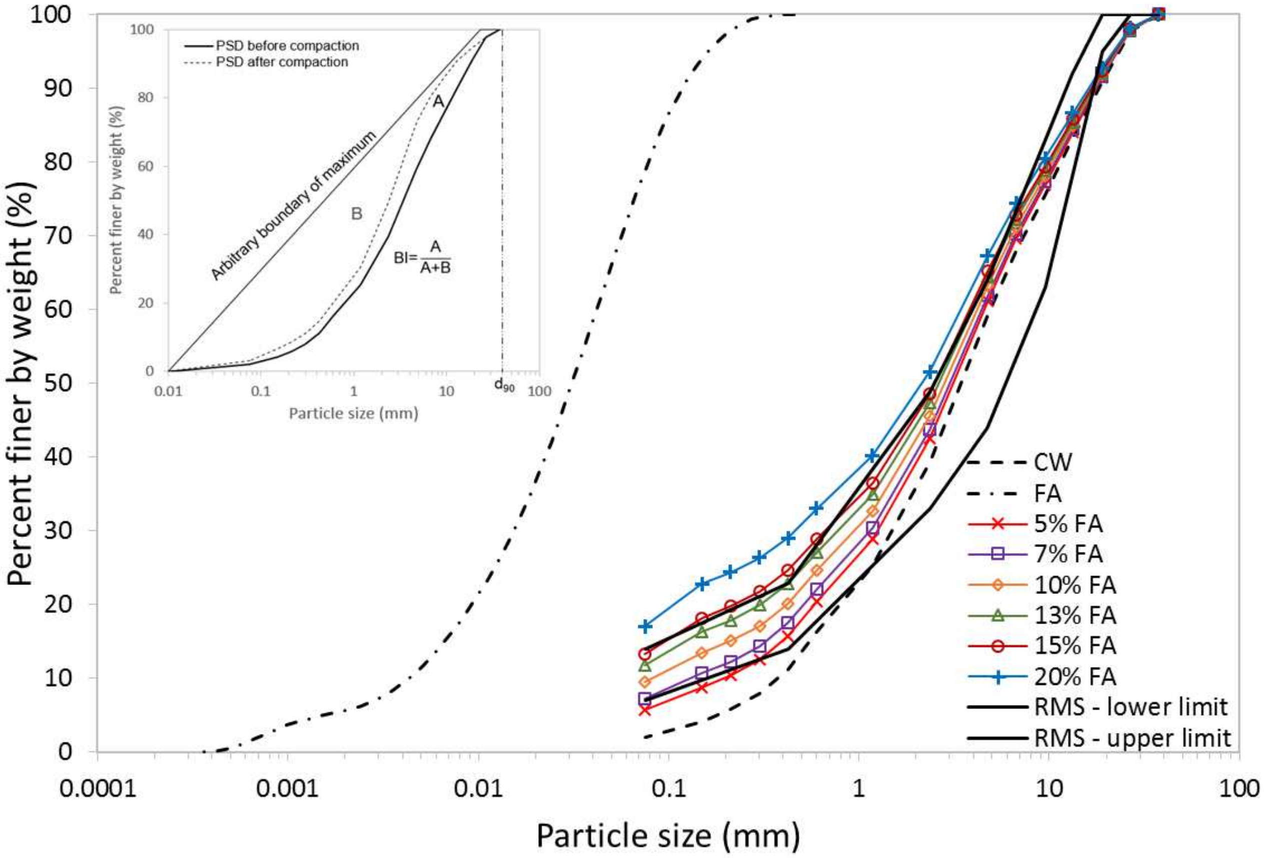

Figure 2: PSD curves of CW, FA and CWFA mixtures

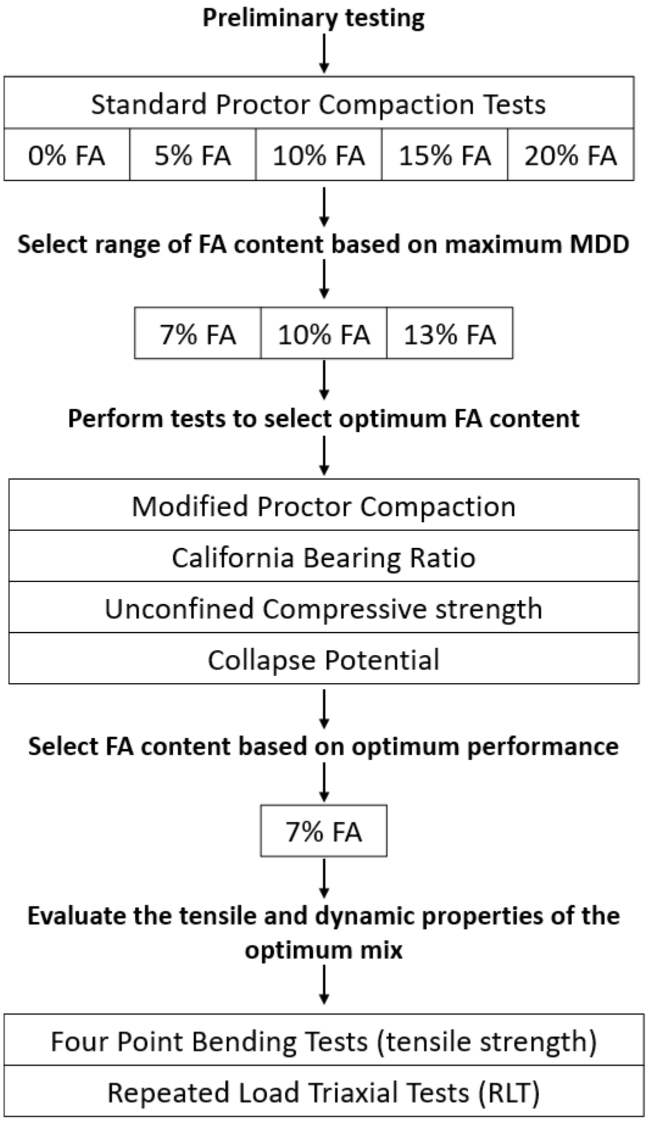

Figure 3: Schematic diagram of the experimental program followed to optimize the CWFA mixture

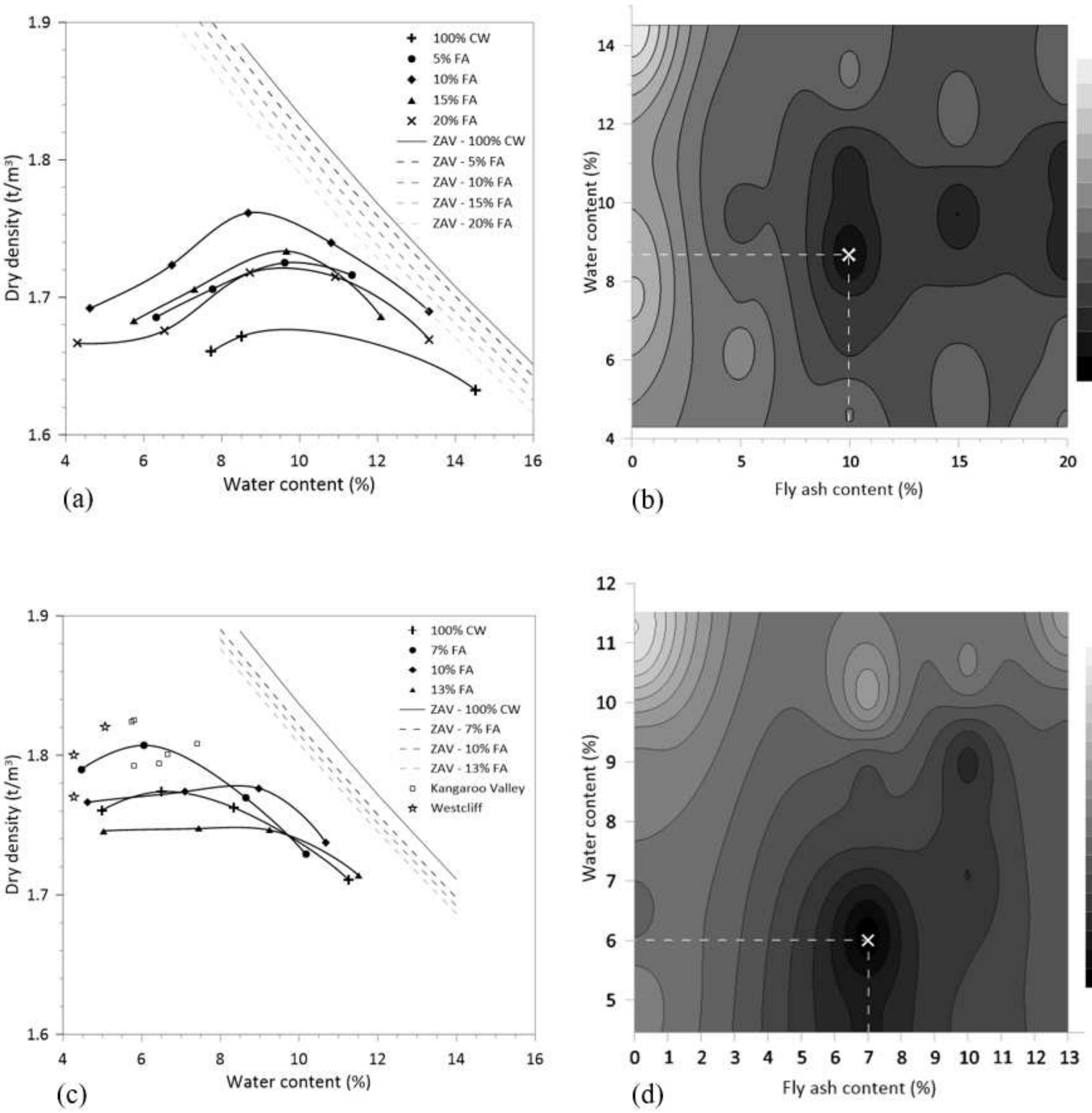

Figure 4: Compaction characteristics at (a-b) standard Proctor and (c-d) modified Proctor

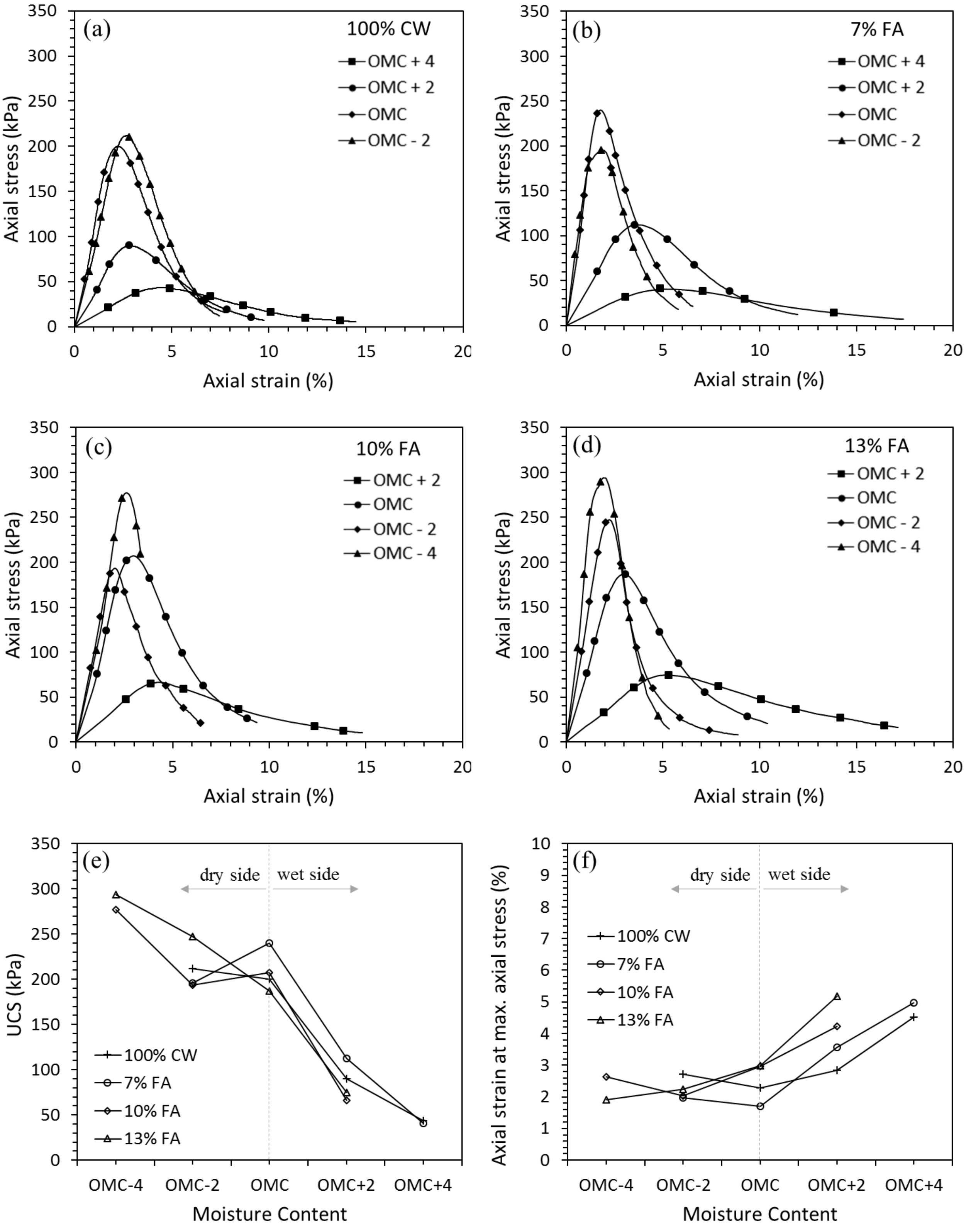

Figure 5: (a) Unconfined compressive strength of 100%CW, (b) Unconfined compressive strength of 7%FA, (c) Unconfined compressive strength of 10%FA, (d) Unconfined compressive strength of 13%FA, (e) variation of UCS with moisture, (f) variation of the axial strain at the maximum axial stress with moisture content

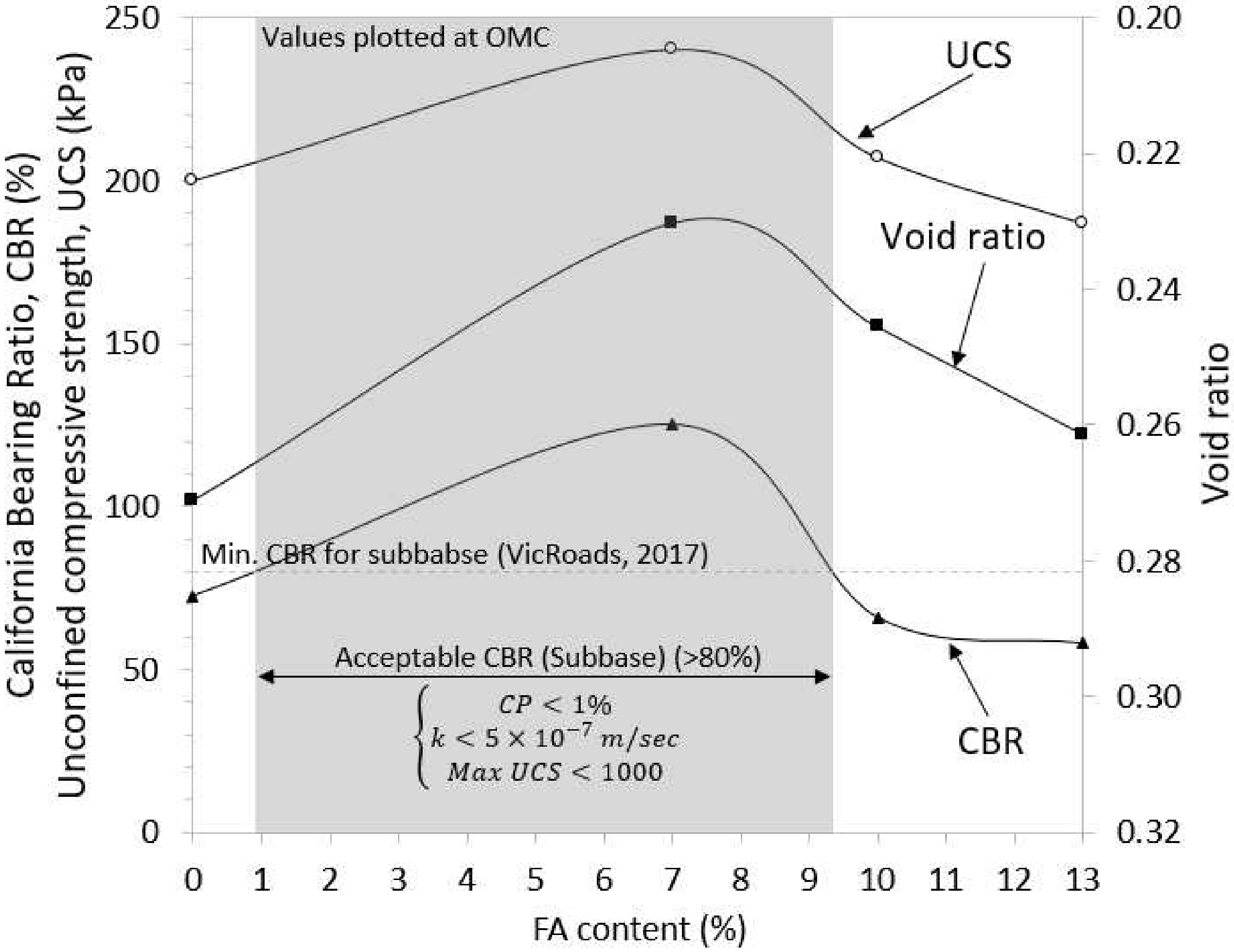

Figure 6: Variations of CBR, UCS and Void ratio under different FA content

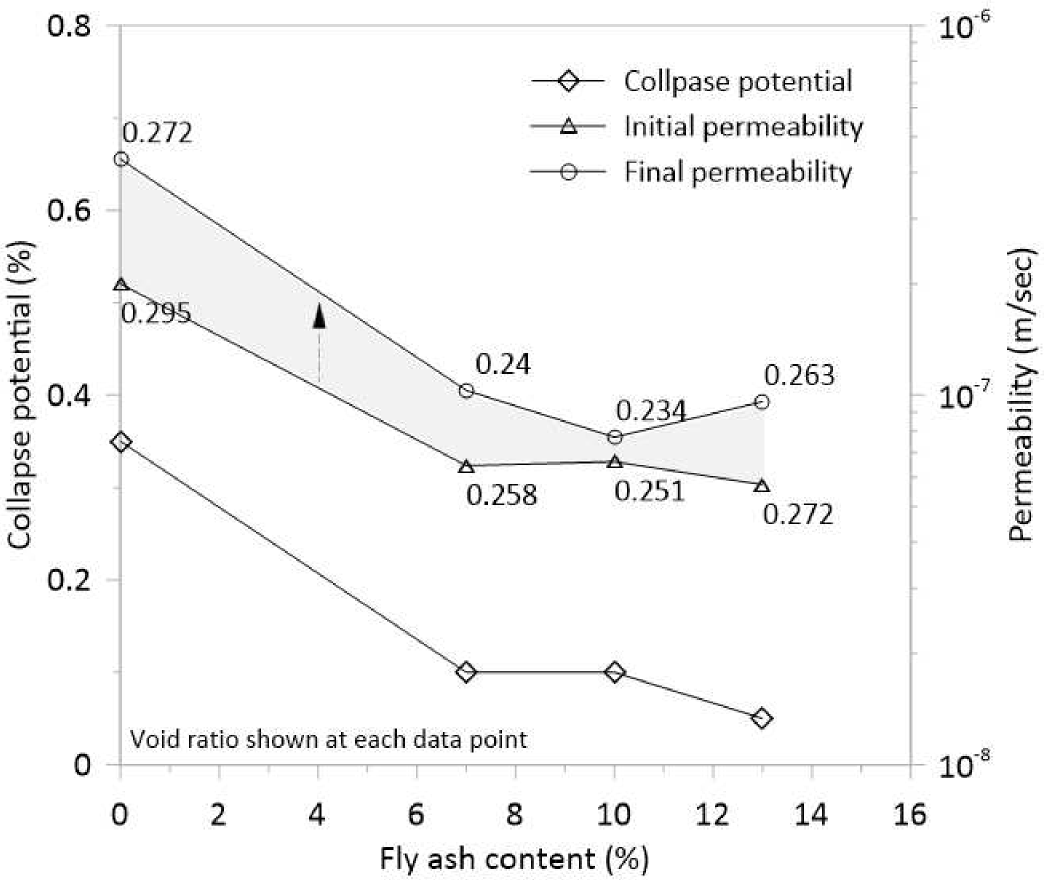

Figure 7: Collapse potential and associated change in permeability

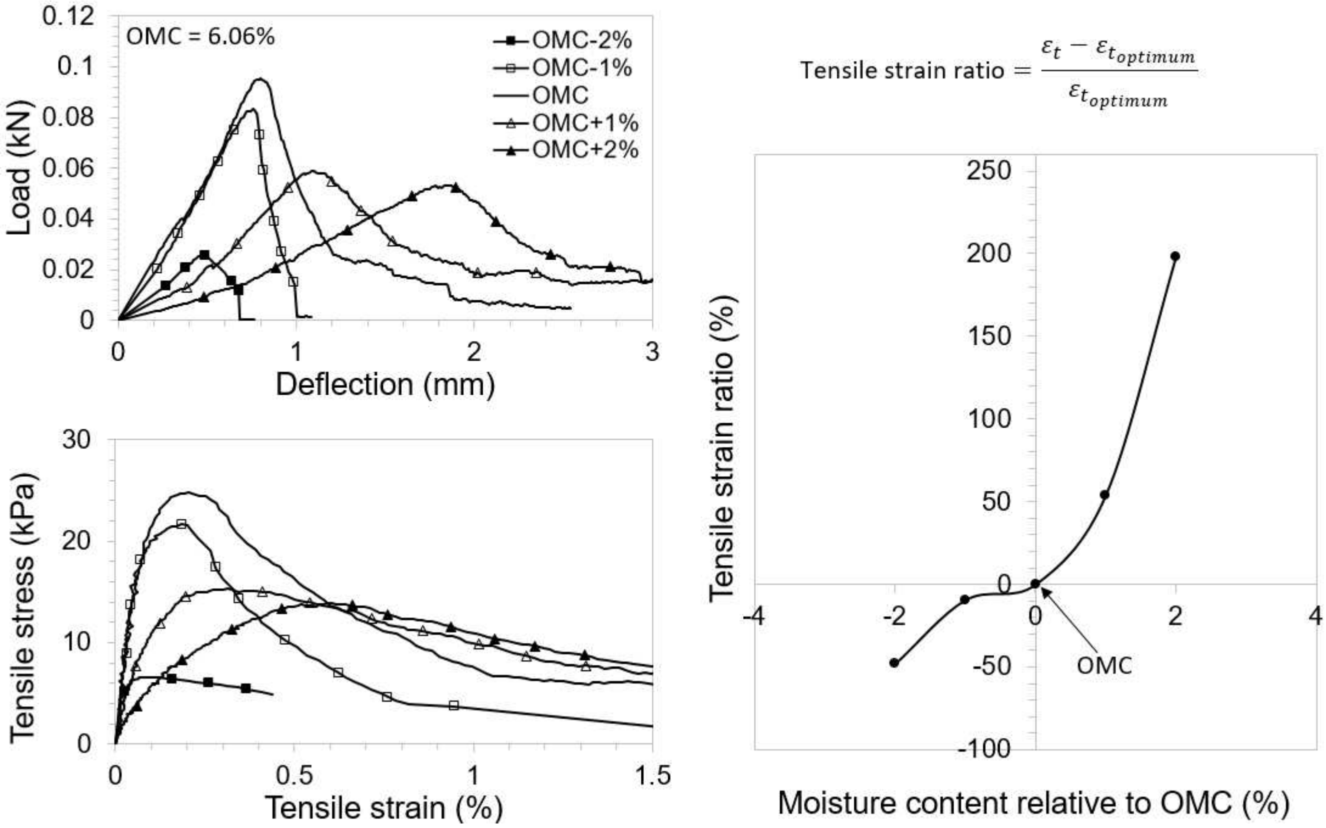

Figure 8: Tensile strength of CWFA mixture with 7% FA and different moisture contents

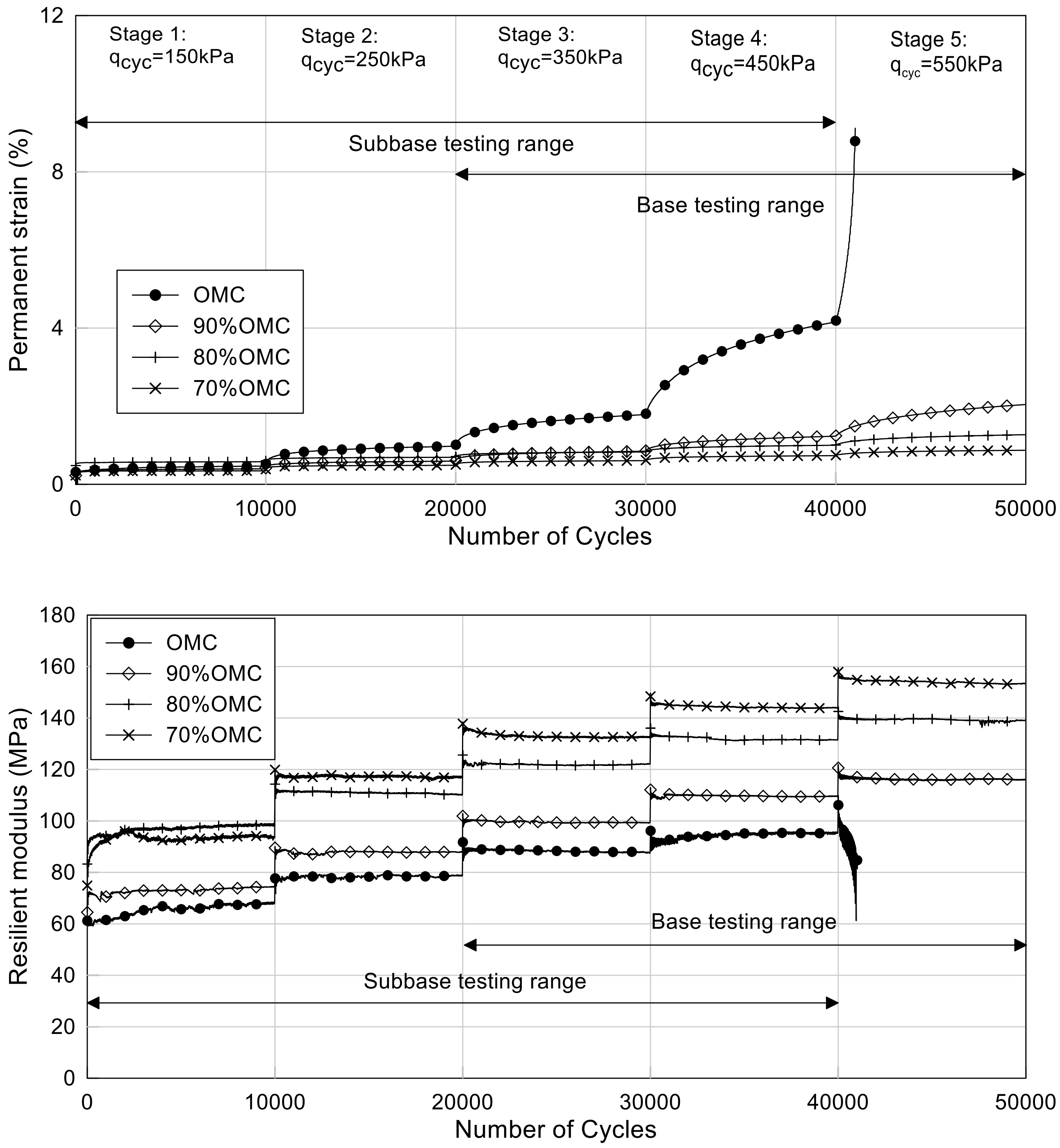

Figure 9: Resilient modulus of the CWFA mixture with 7% FA tested at 50kPa confining pressure for different dry back conditions