Performance evaluation of Construction and Demolition and other waste materials

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Performance evaluation of Construction and Demolition and other waste materials

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ABSTRACT: This paper deals with an experimental study for the characterization of Construction and Demolition (C&D) waste to be used in road construction. The research was carried out both in laboratory, to determine the most suitable mixtures, and in situ, to evaluate their bearing capacity and durability. Relying on the results, different bound and unbound mixtures were then manufactured. To evaluate the on-site performance of these materials, a full-scale experimental field was built in an area of suitable characteristics. The field was composed of a first layer of unbound C&D materials and a second layer of cement stabilized C&D materials. Finally, bearing capacity surveys were carried out using deflectometric devices, and monitoring the density and the moisture of the material. A first analysis of the results shows that both the bound and the unbound mixtures meet the specification requirements in terms of mechanical performance in laboratory and on-site bearing capacity.

1 INTRODUCTION

In the latest EU Directive (2008/98/EC), re-use and recycling processes are encouraged in order to achieve higher levels of recycled waste from both quantitative and qualitative points of view. Article 11 of the Directive states that Member States must take measures to promote high quality recycling of urban waste. In order to comply with these objectives and move towards a European recycling society with a high level of resource efficiency, Member States must take the necessary measures to ensure that (a) by 2020, municipal waste recycling to be increased to a minimum overall level of 50% by weight; (b) by 2020, the preparing for re-use, recycling and other material recovery, including backfilling operations using waste to substitute other materials, of non-hazardous construction and demolition waste to be increased to a minimum of 70% by weight. Italy is still in a backward position in terms of recycling, compared to the very high percentage of recovery in some countries (>90% by weight in the Netherlands, Germany and Denmark) (Fischer et al. 2009). However, in parallel with the growing difficulty of finding natural inert materials and under pressure from these recent regulations, Italy is also taking significant steps forward in this area.

In nature, C&D is extremely variable (Barbudo et al. 2012). This is possibly one of the most important and urgent problems to be solved for the future use of secondary raw materials. Different treatment plants located in different areas will almost always produce very different materials. Each plant will provide a recovered product with particular characteristics. This makes it more difficult to ensure similarities between potentially distant plants. Every system should provide tools and information to support the characterization of its product. Furthermore, such high volume of waste requires strict management in terms of collection, transport, treatment, recovery and final disposal. Current policies aim at changing patterns of the production, consumption and disposal of waste (Thøgersen et al. 2013). Aggregates from demolition along with those produced by the industrial processing of elements, components and prefabricated items are an important secondary source (Molenaar 2013).

Alongside C&D materials, other non-hazardous waste materials can be recycled and used in the preparation of mixtures of aggregates and products for civil construction work, in general. Among these, the most wide-spread are siderurgical waste and the residue from waste-to-energy processes.

The purpose of this research was to evaluate the use of different recycled materials in the production of mixtures for the construction of embankments, foundations and bases. This was achieved through specific laboratory tests and the construction of a full-scale experimental field reproducing the layers of a section of road. The characteristics of the layers' bearing capacity were studied on site, using Continuous Compaction Control and deflectometric (LWD) tests.

2 CONSTITUENT MATERIALS

Two materials were reclaimed from construction and demolition work (Fig. 1) and two from waste disposal plants.

These materials (A, B, C and D) were used to prepared mixtures set in place in a specially constructed experimental field.These materials were obtained from the selective demolition of concrete masonry, correctly deferrized, from the non-selective demolition of buildings and other constructions and from solid urban waste-to-energy processes.

All the base materials, including those from waste disposal plants, were characterized against the main recognized product components. During demolition, materials can be preventively separated at different levels. If the demolition is carried out without this separation, classification tests (UNI EN 933-11) can be used to determine the type and relative proportions of all the constituents in a mixture, in order to establish whether it can be used and if it complies with the regulation standards. The materials are classified according to the European standards set out in Table 1.

Figure 1. C&D from the demolition of selected concrete masonry (left) and non-selective masonry from buildings and other constructions (right).

Table 1. Constituents of non-floating coarse aggregates.

Constituent	Description
Rc	Concrete and concrete products, mortar, concrete walls
Ru	Unbound aggregates, natural stone, hydraulically bound aggregate
Rb	Clay masonry units (bricks and tiles), calcium silicate masonry units, aerated non-floating concrete
Ra	Bituminous materials
Rg	Glass
X	Other: cohesive (clay and soil); Miscellaneous: ferrous and non-ferrous, non-floating wood, plastic and rubber, gypsum plaster.

The test method is specific for recycled coarse aggregates, that is, with a particle size between 4 and 63 mm. The proportion of each material that defines the sample is determined and expressed as a percentage in mass, with the exception of the portion of floating concrete. This is material with a specific weight of less than the specific weight of water, and its relative product portion is expressed as a percentage in volume. Table 2 contains the results of the analysis (FL indicates floating materials).

From the analysis of the four materials used, it emerged that:

- A is recycled cement concrete, that is, aggregates composed mainly of fragments of concrete, including reinforced concrete, from the demolition of reinforced concrete constructions; of these, 90% in mass of the main component is composed of concrete and fragmented lithic material, while 10% is crushed masonry and plaster construction waste;
- B is material from rubble, that is, aggregates composed mainly of concrete and crushed lithic material for 50% or more in mass, and crushed masonry construction waste for less than 50%;
- C and D are composed of the same materials obtained from waste-to-energy processes, in different particle size classes, meaning that product testing is carried out on the largest pieces.

3 LABORATORY TESTS

3.1 *Pre-qualification analysis of materials*

The physical and mechanical characteristics of the recycled materials can be determined using prequalification tests (Table 3).

Specifically, the following tests were performed: Atterberg Limits (UNI CEN ISO/TS 17892-12), Shape Index (UNI EN 933-4), Flakiness Index (UNI EN 933-3), Los Angeles value (UNI EN 1097-2) and Sand Equivalent Test (UNI EN 933-8).

From the results, materials A and B have geometrical properties and properties of resistance to crushing typical of C&D materials.

Table 2. Product analysis, differentiation by type of material (% in weight).

	Rc (%)	Ru (%)	Rb (%)	Ra (%)	Rg (%)	X (%)	FL (%)
A	90	5	5	0	0	0	0
B	50	30	20	0	0	0.1	0.1
C	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.
D	0.0	75.4	1.5	0.0	22.7	0.4	3.0

3.2 *Analysis on unbound mixtures*

Following the pre-qualification analysis of the four materials A, B, C and D, two unbound mixtures were prepared, to be used for the mechanical characterization tests. The composition by weight of the mixtures is the following: M1 (50% A, 50% B) and M2 (70% B, 30% D).

The respective grading curves were determined according to the aggregate gradings for unbound layers set out in the technical specifications being consulted (Autostrade per il Brennero, Autovie Venete, ANAS—the Italian government-owned company for road construction and maintenance).

Table 4 shows the passing percentages by weight for the two mixtures with one of the aggregate gradings of reference.

After this, compaction was assessed using the modified Proctor test, and bearing capacity using the CBR test.

This index was calculated for samples left to cure by air drying for seven days and for saturated samples that were immersed in water for four days after curing in air for seven days after their preparation.

In order to comply with the reference standard UNI EN 13286-2, the modified Proctor test was used to determine the variation in dry density (γd) in function of the humidity present in the sample. Figure 2 shows the results obtained for the two mixtures.

From the tests, it emerged that the optimal humidity is around 10.5% for mixture M1 and 11.0% for M2. Once the optimal humidity and the maximum dry density values were determined for each mixture, CBR tests were carried out on the samples prepared using the optimal parameters. The test was carried out on a sample of six specimens for each mixture, calculating the average as the representative CBR value. The CBR test was performed according to the standard of reference UNI EN 13286-47. Figure 3 also shows the data obtained from the saturated samples.

Following the samples' permanence in water, their CBR values decreased by about 20% for mixture M1 and by about 10% for mixture M2. The variations do not indicate however any decline in bearing capacity such as to advise against their use. Both mixtures are suitable to be used in unbound sub-base and foundation layers, as they are within the lower limit of 50% given in the specifications for pre-saturation tests.

The results obtained by the tests described are shown in Table 5.

3.3 *Analysis on bound mixtures*

Two bound mixtures were prepared, M3 and M4, obtained from M1 and M2, respectively, with the

Table 3. Pre-qualification tests of the constituent materials in the mixes.

	LL (%)	F.I. (%)	S.I. (%)	L.A. (%)	E.S. (%)
A	34.07	13.41	8.24	31.20	37.39
B	34.45	12.41	9.14	35.44	27.36
C	55.27	N.D.	N.D.	N.D.	46.88
D	43.84	17.62	8.50	45.29	57.69

Table 4. Grading curve for M1 and M2 mixtures and grading limits.

%	M1	M2	Min	Max
63	100	100	100	100
40	95	94	84	100
20	84	83	70	92
14	78	79	60	85
8	64	65	46	72
4	50	42	30	56
2	37	29	24	44
0.25	14	12	8	20
0.063	8	7	6	12

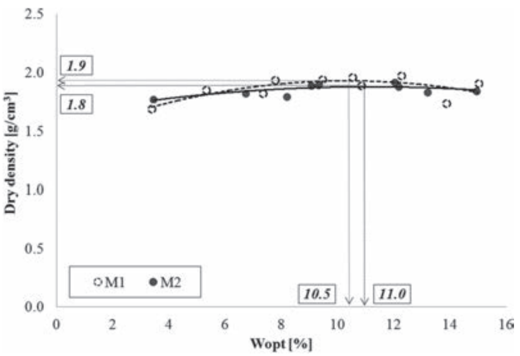

Figure 2. Proctor curve for the two mixtures.

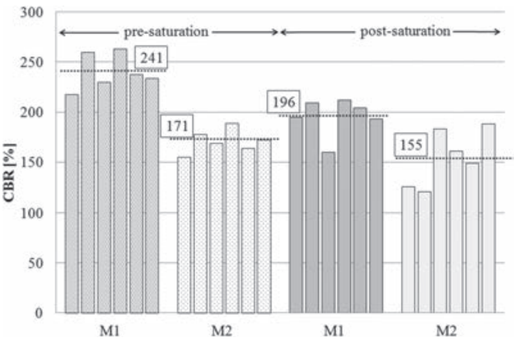

Figure 3. Results of the CBR tests pre and post saturation for mixtures M1 and M2.

Table 5. Proctor Test (UNI EN 13286-2) and CBR Test (UNI EN 13286-47).

Mix	Y_d (g/cm³)	w_{opt} (%)	CBRpre-saturation (%)	CBRpost-saturation (%)	Swelling (%)
M1	1.9	10.5	241	196	0.003
M2	1.8	11.0	171	155	0.003

Table 6. Grading curve for M3 and M4.

%	M3	M4	Min	Max
40	95	94	100	100
31.5	93	91	90	100
20	84	83	70	90
14	78	79	58	78
8	64	65	43	61
4	50	42	28	44
2	37	29	18	32
0.125	10	9	6	13
0.063	8	7	5	10

addition of hydraulic binders, specifically, 1.25% Portland cement 42.5 and 1.25% recycled fly-ash.

Target w_{opt} (%) was obtained adding 1% to the unbound materials corresponding values. The initial grain sizes were set to include the gradings of reference specified for bound mixtures (Table 6).

To obtain the mechanical characterization of bound mixtures M3 and M4, indirect tensile tests and compression strength tests with free lateral expansion (FLE) were performed on the specimens cured in air (wrapped in cellophane) for seven days (Figs 4–6).

The average values for the six specimens are given in Table 7. The Table also shows the average values of indirect tensile tests after 28 days.

Mixture M3 has greater resistance to tension and compression than mixture M4.

In particular, taking as reference values after seven days the values of 2 to 3.5 MPa for the compression test and of 0.25–0.35 MPa for the indirect tensile test, only mixture M3 is totally complaint with the standards, while mixture M4 needs further curing to satisfy the requirements set out for indirect tension.

In terms of the results from indirect tension after 28 days, test values also respect the conditions set out in the technical specifications, that is, a value between 0.25–0.30 MPa and between 0.50–0.60 MPa, respectively.

To evaluate the effect of recycled fly-ash on the mechanical performance, the bound mixtures were prepared using 2.5% cement, that is, without including recycled fly-ash as an integration binder.

Table 8 contains the results of the analysis after seven days' curing.

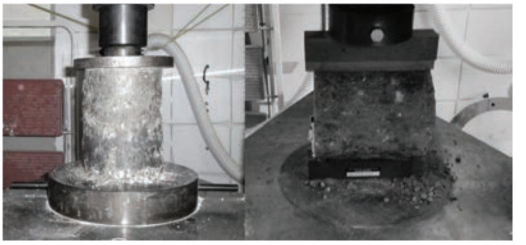

Figure 4. Compression strength test and indirect tensile test for mixtures M3 and M4.

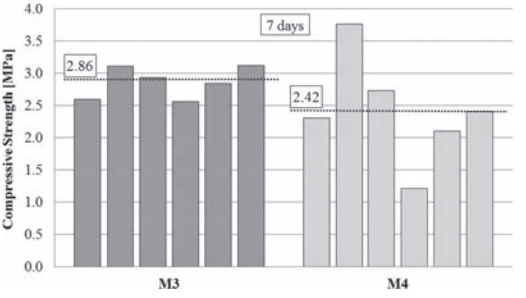

Figure 5. Results of the compression strength tests for M3 and M4, bound to 1.25% Portland cement 42.5 and 1.25% recycled fly-ash.

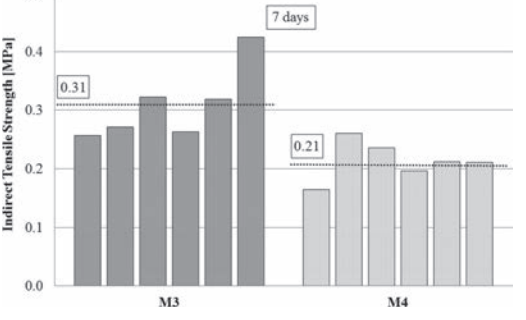

Figure 6. Results of the indirect tensile tests for M3 and M4, bound to 1.25% Portland cement 42.5 and 1.25% recycled fly-ash.

Table 7. Indirect tensile (UNI EN 13286-42) and Compression strength (UNI EN 13286-41).

1.25%Ce 1.25%Fh		w (%)	P (MPa)	Displ. (mm)
CS	M3	11.4	2.86	3.41
	M4	11.9	2.42	3.29
IT	M3	11.4	0.31	0.82
	M4	11.9	0.21	1.02
IT(*28 days*)	M3	11.4	0.35	0.97
	M4	11.9	0.28	0.73

Table 8. Indirect tensile (UNI EN 13286-42) and Compression strength (UNI EN 13286-41).

2.5%Ce		w (%)	P (MPa)	Displ. (mm)
CS	M3	11.9	3.46	3.75
	M4	11.4	3.65	3.18
IT	M3	11.9	049	0.77
	M4	11.4	0.33	0.62

It should be noted that all the mixtures give results that are well above the limits of reference and are suitable to be used for layers bound to cement.

4 EXPERIMENTAL SITE

4.1 *Construction of field*

Constructing a full-scale experimental field was determined by the decision to evaluate on-site the bearing capacity performance of the materials under study, in single and double layers, and to control the evolution of compaction during the construction phases.

The experimental field simulates a road section of 10 m × 30 m constructed with sub-base and foundation. A replacement layer was laid to obtain a homogeneous laying surface and, on that, a first layer of unbound material (M1 and M2) and a second layer of bound material (M3 and M4). The two layers were laid in a suitable geometric configuration, resulting in four different stratigraphic configurations, and these were then used for making comparisons in terms of bearing capacity. All fours fields per layer will be taken into account in the following data analysis (Fig. 7–8).

The sub-base is a layer of replacement material (70–80 cm) composed of natural virgin class A1 material (UNI 10006:2002) with good mechanical and bearing capacity properties.

The replacement was laid so that the existing layer of material, while uniform and naturally packed, could not influence the performance of the materials being studied.

Compaction was carried out on all the layers, including the sub-base, using a vibratory roller with Continuous Compaction Control (CCC) technology, working along four adjacent parallel lanes 2.1 m wide and 30 m long. A concrete kerb was constructed along the centre line of the field at the sub-base level, to assess the measurement depth of the compaction system and study how underlying rigid sections can influence the measurements of the stiffness moduli of the upper layers. The unbound materials M1 and M2 were laid on the sub-base layer at a compacted thickness

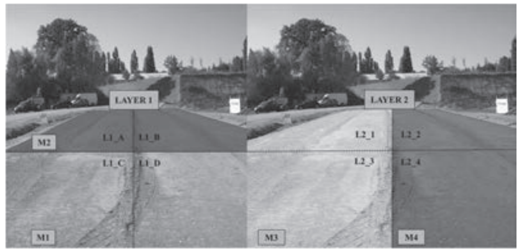

Figure 7. Structure of the experimental field.

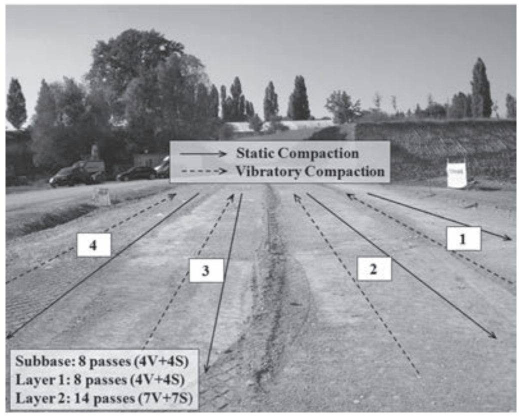

Figure 8. Compaction scheme.

of around 30 cm (layer 1). M1 and M2 were laid after dividing the field crosswise into two surfaces of 10 m × 15 m. The bound materials M3 and M4 were laid over the unbound materials for a compacted thickness of around 25 cm (layer 2). The two symmetric areas are 5 m × 30 m, obtained by dividing the field lengthwise along its centre.

4.2 *Equipment used*

Continuous Compaction Control (CCC) is an important innovation in the construction of roads and has only been introduced recently in Italy. It is based on analyzing the interaction between the dynamic vibratory roller and the compacted material. The compaction set-up is programmed to fix the roller's forward speed and vibration frequency and amplitude (Adam 2007).

At every impulse, the accelerometer system measures the response of the compacted surface, calculating a dynamic stiffness modulus (here identified as Evib), which is used to evaluate the compaction at that pass. A satellite positioning device and an on-board display are used to control and localize the stiffness values measured by the system, which basically becomes a high performance deflectometric device.

On the experimental site, every compacted layer, including the sub-base, was compacted using a Bomag BW213 roller weighing 15 tonnes, with a BCM5 system. Compaction was carried out using a predetermined number of rolls for each layer and setting the vibration amplitude manually, in order to obtain, for that compaction configuration (energy, thickness, humidity) the maximum compaction possible for the materials laid, avoiding any superficial decompaction.

Alongside CCC-type measurements performed during the mechanical compression of the material, a series of deflectometric tests were carried out on site using a Light Weight Deflectometer (LWD), together with density and humidity tests (Fig. 9–10).

The tests were carried out on the replacement sub-base and on the first and second layers, following a regular grid with a total of 24 test points for each lane, 96 for each layer. Further tests were also carried out to determine the level of compaction reached by the materials, by changing the drop set-up (mass and height) of the LWD.

Figure 9. Density tests (a) and bearing capacity tests using a LWD (b).

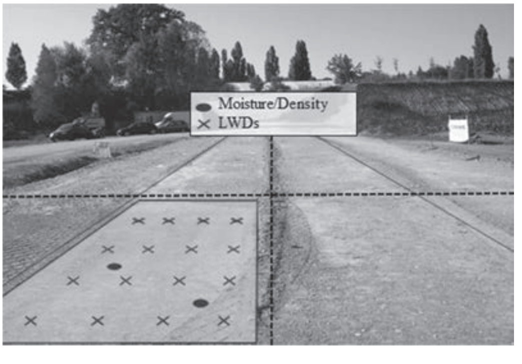

Figure 10. Positioning of the test points for every layer.

Various LWD devices are available commercially. All the instruments provide a dynamic deformation modulus Evd, obtained from Boussinesq's formula and dependent on the structure of the LWD.

The LWD Good Practice Guide identifies two main categories (Edwards et al. 2009), both used to measure the experimental field.

- Category 1 (C1): these allow variations to the fall height of the drop rammer, the deflections can be measured using a geophone placed in direct contact with the surface through a hole in the centre of the plate and there is a load cell to register the time histories of the impulses below the plate (ASTM E2583);
- Category 2 (C2): these have a fixed fall height, ground deflection is measured using an accelerometer placed between the buffer and the load plate to measure the plate deflection and there is no load cell (TP BF-StB Part 8.3, ASTM E2835).

The C1 device used includes a load cell and it is possible to set a specific load by fixing the fall height according to the type of layer being examined. The C2 device used sets a nominal top force (calibrated by the manufacturer) equal to 7.07 kN for a mass of 10 kg and 10.60 kN for a mass of 15 kg.

It is presumed that these forces act on the top of the plate and that they are assumed to be constant in the calculation of vertical pressures under the plate. The impulse is applied with load curves of a mainly semi-sinusoidal form that differ by length of impulse. The period of the curve of C2 is roughly 16 to 18 ms, while that of C1 is roughly 28 to 30 ms (Sangiorgi et al. 2009).

4.3 *Analysis of on-site density/moisture tests*

Four density and moisture tests were carried out on-site (ASTM D1556), for each field of every layer, to compare with the dry density and optimal moisture obtained using the laboratory Proctor test. The results are given in Table 9.

The level of density reached on site for the various layers on the various fields is satisfactory and greater than 90–95% of the corresponding values of maximum dry density obtained from the Proctor compaction study.

4.4 *Analysis of CCC tests*

Using CCC compaction, it is possible to determine the stiffness properties acquired by the material during the construction phases in relation to the stratigraphic coupling of bound and unbound materials, and so determine the compaction achieved and its uniformity.

Table 9. On-site density and moisture tests.

Fields	Sub-base (kg/m^3)	Layer 1 (kg/m^3)	Layer 2 (kg/m^3)
1	1969	1944	1957
2	1968	1960	1954
3	1972	1940	1944
4	1958	1947	1960

We will first present the analysis for the unbound materials M1 (50% A, 50% B) and M2 (70% B, 30% D) on layer 1 (L1). The first area is formed by fields L1-C and L1-D (material M1) and the second by fields L1-A and L1-B (material M2).

To evaluate the compaction evolution, a comparison was made between the various average moduli values between one vibratory pass and the next, tracing the splitted lines identified as the Compaction Paths (CP) obtained by joining these values (Sangiorgi et al. 2012).

Figure 11 shows the CP of the tests performed in the zones L1-C and L1-D, material M1. The mean values of the Evib data obtained for the four lanes at each single pass were plotted for each field. If the values between the following passes are close to each other, the average value of the stiffness moduli approximates to the equivalence line at 45°.

Optimal compaction, defined as the maximum stiffness that can be achieved in any one compaction configuration (roller-layer), is obtained when two successive passes do not register any variation in modulus.

The mean Evib modulus value is practically constant (around 46 MPa) when the number of passes increases and there is an increase of only 2 MPa between the first and last pass. This indicates that the compaction action does not determine increases to stiffness in the compacted layer.

The behaviour of mixture M2, for zones L1-A and L1-B is equivalent to that of M1 and optimal compaction is already achieved with the first two vibratory passes, with a total increase of only 2 MPa.

To give a better reading of the uniformity of Evib data, mappings can be presented where all the non-sampled values are estimated through interpolation. In fact, the representation of stiffness values using mapping provides a simple tool to interpret the compaction action of the roller and the uniformity of the values (Dondi et al. 2013).

In this case, the estimated mean Evib value for basic surfaces of 2.1×0.1 m^2, where the surface of the field can be discretized, essentially coincides with the mapping of the sample itself.

As can be seen in the mappings in Figure 12, the stiffness of materials M1 and M2 is between 45 and 55 MPa and is uniformly distributed across the entire experimental site.

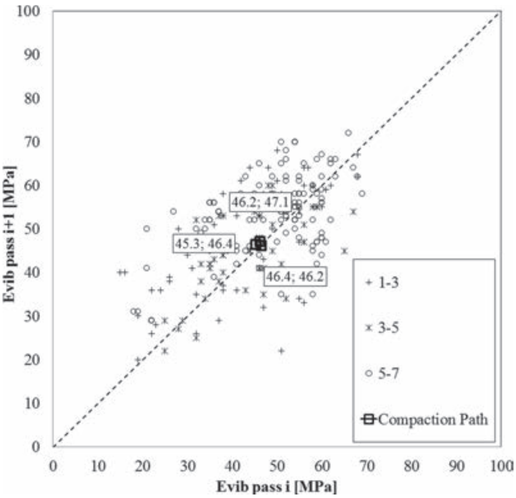

Figure 11. Compaction Path for fields L1-C and L1-D.

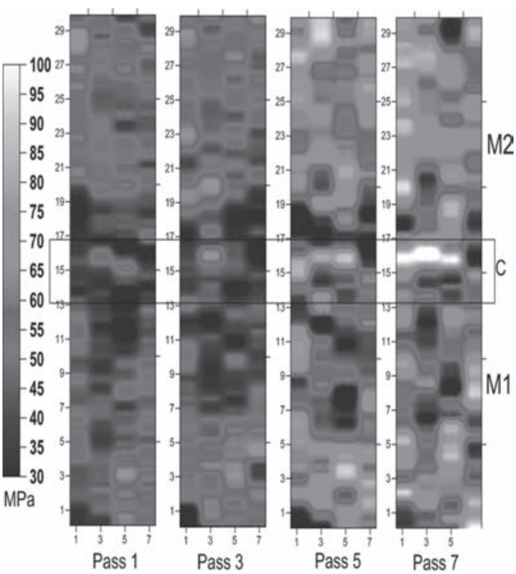

Figure 12. Mappings of the Evib values of L1 for each pass.

The bound materials M3 and M4 on layer 2 (L2) behave differently compared to the unbound materials of the lower layers.

The increase in number of passes involves a constant increase of the value of the Evib modulus, which reaches peaks of above 90 MPa, without taking into account the zones affected by concrete kerb in the sub-base.

From the analysis of the data, it is possible to observe a distinct difference in behaviour between the internal and the external lanes of each lane for each of the mixtures used, with an increase in stiffness of the structure in the more confined central zones compared to the external lanes. Only the analysis of field 1 is presented, as the behaviour of the other fields is analogous.

In the graphs in Figures 13 and 14, the initial Evib moduli are similar (50 MPa), but the CPs are different leading to stiffness values that differ according to zone. They are above 85 MPa for the internal lane and are equal to 60 MPa for the external lane.

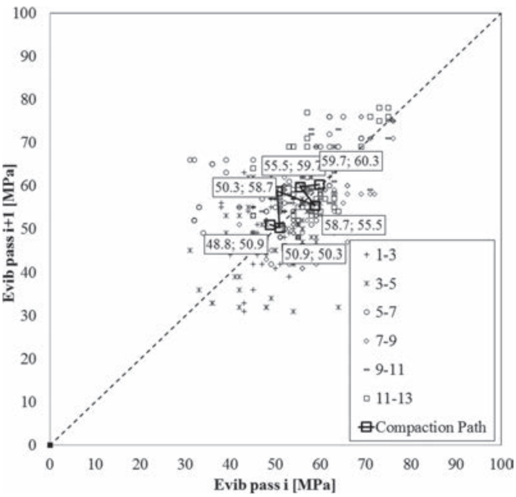

Figure 13. Compaction Path of the area L2–1 lane 4 external.

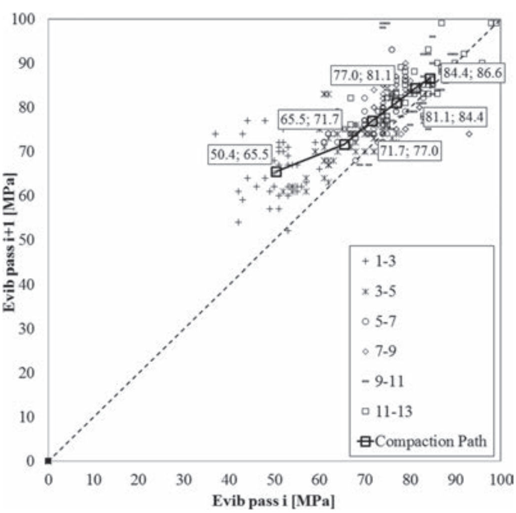

Figure 14. Compaction Path of the area L2–1 lane 3 central.

The CPs show how the evolution of stiffness of the laid material is different between internal and external lanes, in particular the central lane evolves with greater continuity approximating the equivalence line, while, for the external lane, progress is irregular and subject to decreases in stiffness.

The same behaviour appears for all four fields of layer 2 (maps in Fig. 15). In the maps, the progress of stiffness of the entire field clearly increases as the numbers of passes follow each other, showing particularly the behaviour of the two central lanes.

Assuming that the central lanes are not affected by interference due to the construction phases of the field, the bound mixture M3 has higher stiffness values than mixture M4. Specifically, the set M1-M3 registers a dynamic Evib modulus that reaches peaks of above 85 MPa (assuming that the central zone is not affected by the kerb in the subbase).

Figure 16 also gives the mean values by pass and for each layer examined.

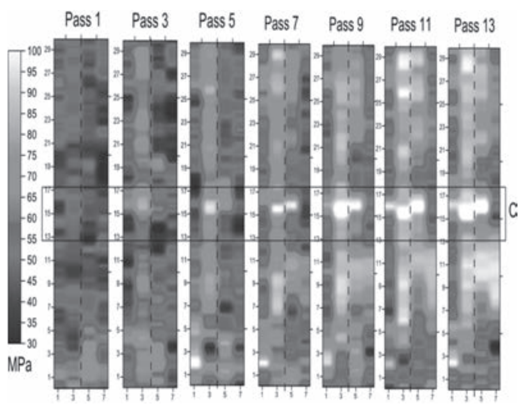

Figure 15. Mappings of the Evib values of L2 for each pass.

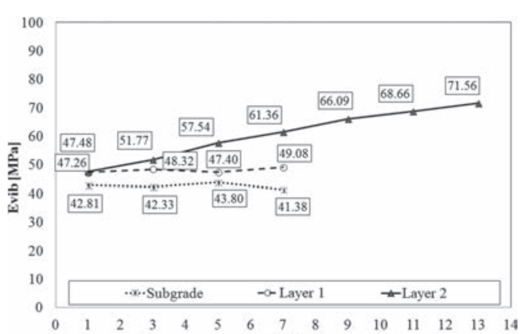

Figure 16. Progress of the average Evib value per layer.

The bound mixtures M3 and M4 on layer 2 highlight a constant increase of the dynamic modulus with the increase in number of passes, until reaching average values of between 70 and 80 MPa.

The unbound mixtures M1 and M2 used in the layer L1 of a thickness of 30 cm show modest percentage variations of Evib and, on average, stiffness remains on values only slightly above those obtained for the replacement layer.

4.5 *Analysis of LWD data*

In order to control the bearing capacity performance of the tested materials, below is the analysis carried out on the layers using the LWD class C1 deflectometric system.

The mappings of Evd moduli shown in Figure 17 are calculated by interpolating the experimental data obtained from the survey grids described above (96 measurement points).

The instrument is configured as: falling weight of 15 kg, plate radius 150 mm, *f* equal to 1.57, ν equal to 0.35 and fall height and consequent load peak calibrated so that a stress of 130 kPa is applied, independently of the material being tested. The choice is dictated by the decision to obtain a single comparison parameter between the different materials.

Comparing the moduli obtained from the test using LWD-C1 on the three superimposed layers, it can be seen that, similarly to the values obtained from the CCC data, the modulus values of mixtures M1 and M2 are comparable, or even slightly lower (M2), to the values obtained for the replacement sub-base layer in natural gravel (Fig. 17).

Mixtures M3 and M4 present high moduli, reaching, in the central areas of the compacted sections, values suitable for bound foundation layers.

Coherently to the CCC representations, the mapping being used highlights, although in a less evident manner, that the stiffness are more accentuated in the central sections of the experimental field.

Figure 18 makes the comparison between the Evd value mediated among the three test points on the single pass on all layers and the corresponding Evib value in the area surrounding the test point.

The evident connection that exists between the data obtained from the two devices confirms the goodness of the experimental and regulatory applications that use light deflectometers to control the results obtained from CCC systems and vice versa (White et al. 2007).

In Italy, the ANAS specifications only allow for LWD tests to control compaction of the layer to determine whether it is suitable. The LWD model used in the controls is of type C1, with a mass of 10 kg and peak pressure equal to 70 kPa.

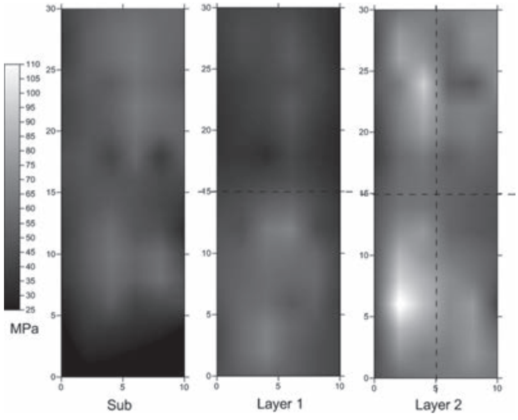

Figure 17. Mapping of Evd values for a class C1 LWD.

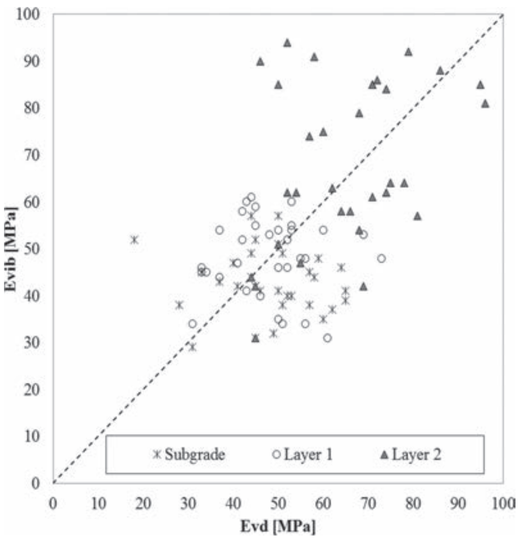

Figure 18. Evd-Evib comparison.

With reference to the standard ASTM E2583, the measurement drops must be repeated until registering a difference of the deflection value between the next two drops of less or equal to 3%. The ANAS procedure also prescribes that, considering test points of at least 5 m apart, this condition must not be met after more than four drops for more than five test points.

Several prior studies (Marradi et al. 2011) have confirmed that the variance in measurements between the successive deflection measurements, to evaluate the quality of compaction, can be divided into the following classes:

- 1.0% ÷ 2.5% Excellent,
- 2.5% ÷ 3.0% Good,

– 3.0% ÷ 3.5% Moderate,
– >3.5% Poor

where the variance% indicates the differences in deflection reached after four drops.

Table 10 shows the evaluations obtained on layer 1 and 2 with 4 control measurements for every field for a total of 16 measurements.

The percentage variance between the deflections measured in two successive drops is always below 3.0% within the fourth drop. As can be seen from the results, the standard is respected and the layer does not require further compaction or replacement.

Having verified the quality of compaction, we then proceeded to compare the values measured using the values imposed by ANAS.

For a foundation or sub-foundation layer in mixed granular stabilized materials, *"[...] the bearing capacity of the layer must be observed using a Dynatest-type LWD with a minimum value of 80 MPa [...]"*. For a foundation layer in cement bound material, *"[...] the bearing capacity must be observed using a Dynatest-type LWD, with minimum values of 60 MPa, after 4 hours, and 200 MPa after 1 day [...]"*.

Table 10. Results—ANAS procedure: compaction.

	Layer 1		Layer 2	
Field—Lane	Var%	Assessment	Var%	Assessment
1–3	2.92	Good	0.45	Excellent
	0.34	Excellent	0.35	Excellent
1–4	1.09	Excellent	3.32	Moderate
	1.39	Excellent	1.75	Excellent
2–2	0.37	Excellent	0.00	Excellent
	1.59	Excellent	0.75	Excellent
2–1	2.36	Excellent	1.50	Excellent
	0.38	Excellent	1.60	Excellent
3–4	1.86	Excellent	1.02	Excellent
	0	Excellent	0.61	Excellent
3–3	0	Excellent	2.49	Excellent
	2.01	Excellent	1.10	Excellent
4–1	0.93	Excellent	0.39	Excellent
	1.21	Excellent	3.18	Moderate
4–2	1.42	Excellent	2.87	Good
	2.08	Excellent	1.84	Excellent

Table 11. Results of the ANAS procedure: stiffness.

field L1	A	B	C	D	Req. (MPa)
field L2	1	2	3	4	
subgrade	76	91	67	65	
layer 1	64	69	85	108	>80
layer 2	72*	68*	81*	60*	>60 4h

*after 2 hour

The values in Table 11 are mediated over four observations and divided by field on ever layer.

In terms of the unbound materials (layer 1) and, as a confirmation of the results obtained from laboratory tests, the highest-performing material is M1, which is mostly composed of concrete (main component of materials A and B). The use of 30% material D in mixture M2 leads to a decrease in bearing capacity of the material given by the modulus Evd, observed in the external lanes of fields L1_A and L1_B. The stiffness of the layer bound to cement after two hours' curing can be considered satisfactory for all the fields.

5 CONCLUSIONS

In the light of the laboratory examinations and the data acquired on the experimental field, it can be stated that:

– the base materials used, obtained from a focused recycling process, if correctly selected and mixed have the capacity of giving, in general, a performance similar to that of virgin aggregates in forming sub-bases and foundations of road pavements;
– the mechanical tests in laboratory indicate that the materials proposed satisfy the requirements on the main standards and that recycled binders can be conveniently used in bound mixtures;
– on site, the proposed unbound mixtures show a certain tendency to compaction, reaching maximum CCC stiffness values after only a few roller passes; the LWD modulus values indicate that these materials satisfy the performance requirements of the ANAS specification for road pavements;
– on site, bound mixtures, where part of the binder is also recycled, give significant bearing capacity values. CCC stiffnesses evolve continuously and are established at Evib values on average above 70 MPa. LWD data show that even after two hours from compaction, these layers reach moduli that satisfy the specifications;
– stratigraphic terms, determined in this case through the CCC data, it can be observed how the coupling of mixtures containing mainly concrete (M1 and M3) gives higher bearing capacity values than the other stratigraphic sections; the M2-M4 stratigraphy is less stiff;
– with reference to compaction and the controlling of production, it can be affirmed that the use of a CCC system can contribute significantly to the processes of control both in terms of time and in terms of data representativeness, allowing, during construction, the monitoring of compaction, stiffness and uniformity of compaction.

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