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Railway ballast anisotropy testing via true triaxial apparatus

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Abstract

This paper aims to demonstrate the highly anisotropic behaviour of railway ballast via true-triaxial tests. To do so, a novel, large-scale, true-triaxial testing apparatus (GeoTT) is designed and constructed. It consists of six hydraulic actuators, designed to apply a distributed stress to large granular cubic test specimens with dimensions: 500mm × 500m × 500mm. To show the capability of the new facility, crushed granite railway ballast with d_{50}=43mm is tested. Three different confining stresses are applied to determine the Poisson's ratio and modulus in three dimensions. Anisotropic behaviour is clearly evident, with horizontal directions showing a lower modulus compared to the vertical direction. It is also found that confining stress has an important effect on both Poisson's ratio and modulus when primary loading is applied in three orthogonal directions. These results are useful for understanding the behaviour of railway ballast and for the calibration of railroad numerical models.

Key words: Granular particle anisotropy; Railway ballast; true triaxial testing (GeoTT), modulus; Poisson's ratio; Railroad

1 Introduction

Granular soils are often referred to as aggregates and are a common construction material for pavements and railways. A large number of studies have been undertaken to quantify the isotropic behaviour of granular particles, including (Roscoe et al., 1963; Lade and Duncan, 1975; Van Eekelen, 1980; Sagaseta, 1987; Alonso et al., 1990; Laloui, 2003; Rotta et al., 2003; Alonso et al., 2012). Testing of granular materials with large maximum particle size requires larger-scale testing apparatus compared to the testing of smaller particles. This is because larger sample volumes are required to ensure the ratio between maximum particle size and sample dimension is low.

Although isotropic loading tests provide insights into material behaviour, many granular materials actually behave in an anisotropic manner (Miura et al. 1984, Tutumluer 1995, Tutumluer and Thompson 1997, Tutumluer and Kwon 2009). Table 1 outlines a range of studies performed using traditional triaxial cells to explore anisotropy of response. The anisotropic behaviour of ballast is important for railways because it aids the understanding of ballast degradation and thus maintenance costs. In particular, it is an important material input when numerically modelling ballast.

However, investigating anisotropy ideally requires the test sample to be subject to a range of stress paths that are difficult to achieve using standard triaxial testing. Therefore, the two most common approaches to achieve this are: Hollow Cylinder Apparatus (HCA) and true triaxial (TT) testing. HCA methods are useful for simulating the rotation of principal stresses that occur during wheel passage. Alternatively, TT methods account for the effect of intermediate principal stresses, which in reality, may be different from the minor principal stresses when considering full anisotropy.

HCA works by subjecting a hollow, cylindrical soil sample to an axial load and torque about the central vertical axis, while applying external and internal radial pressures. The torque results in shear stresses while the axial load combined with the radial pressures results in vertical stress (Cooling and Smith, 1936, Hight et al., 1983, Saada, 1988, Grabe, 2003).

The majority of HCA research into granular particle anisotropy to-date has focused on materials with relatively small maximum particle size (see Table 2). For example, Tatsuoka *et al.* (1986), Pradhan et al. (1988a) and Pradhan et al. (1988b) tested soil specimens with inner diameter, outer diameter and height of 60mm, 100mm and 200mm, respectively. They investigated the strength and deformation properties of Toyoura sand. Alternatively, Yang et al. (2007) used larger HCA apparatus to investigate the anisotropic behaviour of saturated sand. Alternatively, Lade, Nam and Hong (2008) and Lade (2008) used HCA tests to study the cross-anisotropic behaviour of Santa Monica beach sand and found that cross-anisotropy correlated with increasing inclinations of the major principal stress direction. O'Kelly and Naughton (2005), O'Kelly and Naughton (2009), Yang (2013), Yang et al. (2016) and Rolo (2004) also used HCA testing to investigate the anisotropic behaviour of sands.

As an alternative to HCA testing, TT testing works by subjecting a soil sample to stresses in the three orthogonal planes, often using two hydraulic actuators in each plane, either via rigid flat plates or flexible membranes or a mixture of both (see Table 3 for a summary of previous studies). Selig et al. (1979) and Desai et al. (1983) developed true triaxial test setups to apply a three-dimensional, independently controlled, and compressive stress state, using fluid or pneumatically pressurized flexible cushions to transmit stresses in three orthogonal directions, to a cubic sand-ballast specimen with dimensions 101.6mm × 101.6mm × 101.6mm. Isotropic loading was applied to specimens to determine anisotropic response behaviour (i.e., directional dependencies of compacted specimen responses). Alternatively, Yamada and Ishihara (1979) used true triaxial apparatus with a cubic sand specimen of dimensions 100mm × 100mm × 100mm. Results indicated that behaviour was highly anisotropic, inherently due to grain orientation, size and shape. However, as the applied shear stress increased, at failure, the inherent anisotropic effects disappeared.

Alternatively, Reis et al. (2011) developed a cubic triaxial cell to test 60mm specimens of saturated and unsaturated soil. Further, Ochiai and Lade (1983) used true triaxial apparatus to study the anisotropic behaviour of Cambria sand and found that the major principal strain was the lowest when the dilation rate was at a maximum. The same apparatus was then used to develop a failure criterion for cross-anisotropic soils (Abelev and Lade 2003, Abelev and Lade 2004).

Furthermore, Tutumluer and Seyhan (1999) and Seyhan and Tutumluer (2002) used a triaxial device to test aggregate samples with 150 mm diameter and 150 mm height. The vertical modulus was found to be larger than the horizontal modulus for all tested aggregates except one gravel specimen which contained 16% fines (defined as passing the No. 200 sieve or smaller than 0.075 mm) in a dense-graded base course aggregate with a maximum size of 25 mm.

When testing granular particles, it is important to maximise the sample size-to-particle ratio, defined as the minimum dimension of the test sample divided by maximum particle size. If too small, individual particles dominate test results thus causing testing errors. As a guide, Nitchiporovitch (1969) and Fagnoul and Bonnechere (1969) suggested a minimum sample size-to-particle ratio of 5, while Marachi et al. (1972) proposed a ratio of 6. Therefore, because the width of the HCA wall is relatively thin, it is not well-suited for testing large diameter particles. True triaxial apparatus is arguably better suited because it can house a cuboidal volume of granular material, with potentially larger dimensions than the HCA. However, even then, it is challenging to construct a TT apparatus of sufficient scale to investigate anisotropy of samples containing large granular particles.

This paper addresses these sample size challenges by developing a new TT facility capable of testing soil samples with dimensions: 500mm x 500mm x 500mm. The large potential test volume means it is well suited to testing large-particle granular soils, including railway ballast. The maximum particle size tested in this study was 63mm, giving a sample size-to-particle ratio of approximately 8. The facility was used to apply tri-directional stress patterns to railroad ballast and investigate its anisotropic behaviour. Since the previous research was focused on the study between confining stress and Poisson's ratio in the vertical direction only. This paper extended this concept and revealed the relationship between confining stress and Poisson's ratio in the horizontal direction.

Table 1. Anisotropic tests conducted using traditional triaxial cells

Source	Soil type	Dimension (mm)	Aim

Miura, Seiichi and Toki (1984)	Sand	Diameter =70mm, Height = 170mm	Anisotropy, stress-strain curves, liquefaction
Tutumluer and Seyhan (1999) and Seyhan and Tutumluer (2002)	Aggregate	Diameter =150mm Height =150mm	Anisotropy, resilient behaviour
Rolo (2004)	Sand/clay	Diameter =100mm Height =200mm	Anisotropy, shear strength
Aursudkij, McDowell and Collop, (2009)	Ballast	Diameter =150mm Height =450mm	Resilient modulus and Poisson's ratio
Ngo, Indraratna and Rujikiatkamjorn (2017)	Ballast	Diameter =300mm Height =600mm	Anisotropy behaviour, mobilized friction angle

Table 2. Anisotropic tests conducted using hollow cylinder apparatus

Source	Soil type	Inner diameter/ Outer diameter/ Height (mm)	Aim
Hight, Gens and Symes (1983),	Sand/clay	203/254/254	Principal stress rotation effects
Tatsuoka *et al.* (1986),	Sand	30/50/200	Anisotropy, shear strength
Pradhan, Tatsuoka and Horii (1988), Pradhan, T.B.S., Tatsuoka, F. and Horii (1988)	Sand	60/100/200	Anisotropy, shear strength
Grabe (2003)	Sand	60/100/200	Principal stress rotation effects, anisotropy
Rolo (2004)	Sand/clay	76/100/200	Anisotropy shear strength
Yang, Li and Yang (2007)	Sand	150/314/200	Anisotropy, intermediate principal stress
Lade, Nam and Hong (2008) and Lade (2008)	Sand	180/220/400	Principal stress rotation effects, anisotropy, Shear strength
O'Kelly and Naughton (2005) and O'Kelly and Naughton (2009)	Sand	71/100/200	Anisotropy, small strain, yield criterion
Yang (2013), Yang et al. (2016)	Sand	60/100/200	Anisotropy, plasticity, non-coaxiality

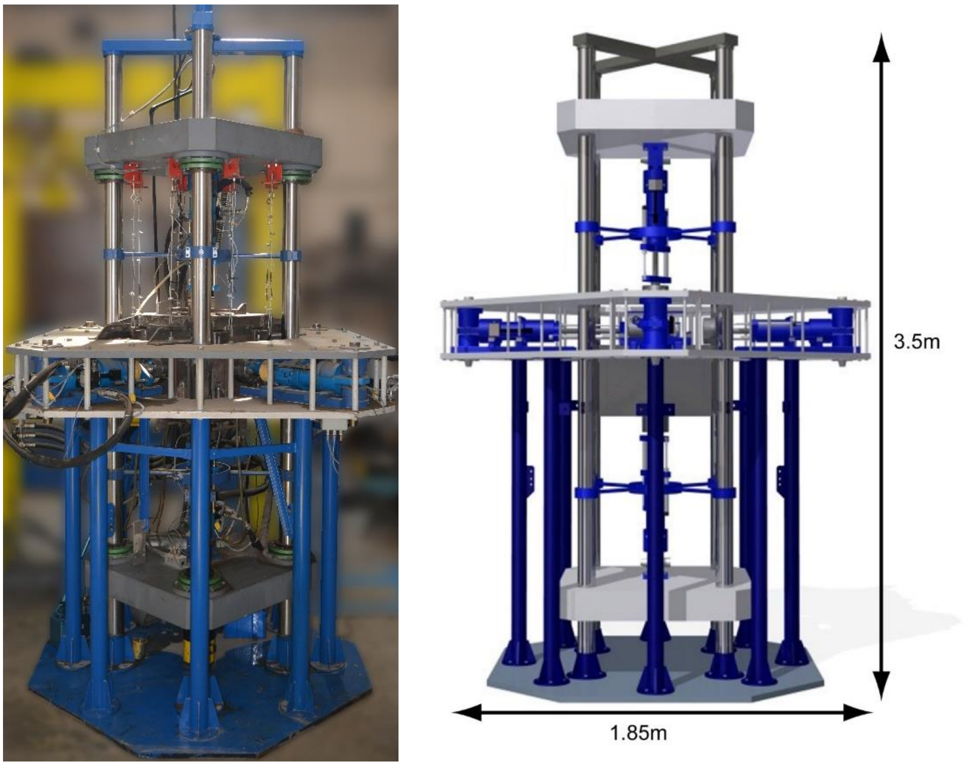
Table 3. Anisotropic tests conducted using true triaxial test apparatus

Source	Soil type	Dimension of cubic sample side length (mm)	Aim
Yamada and Ishihara (1979)	Sand	100mm	Anisotropy, shear strength
Selig, Sture and Desai (1979) and Desai, Siriwardane and Janardhanam (1983)	Sand/ballast	101.6mm	Anisotropy, stress-strain curves
Ochiai and Lade (1983)	Sand	76mm	Anisotropy, stress-strain behaviour
Reis et al. (2011)	Sand	60mm	Anisotropy, saturated and unsaturated
GeoTT (present project)	Railway ballast	500mm	Anisotropy, Poisson's ratio, modulus

2 Apparatus development

2.1 True triaxial test rig

A true triaxial testing facility (hereafter called 'GeoTT') was designed for large granular particle testing in collaboration between Heriot Watt University and The University of Glasgow. It is 3.5m high and 1.85m wide, with the ability to house test samples with maximum lateral dimensions of 580mm (Figure 1). It consists of 6 independent hydraulic actuators, with 2 aligned in each Cartesian plane, making it well-suited for the large-scale testing of anisotropic behaviour (Figure 2). Also, using 6 rams instead of 3 means that a more uniform stress distribution can be applied to test samples. Thus, the effect of varying confining stress can be investigated. Each ram is connected to a load cell and a linear variable displacement transducer (LVDT) for control purposes. The control setup allows for a wide range of independent signal types to be fed into each ram.



(a) (b)

Figure 1. True triaxial testing apparatus: (a) Photograph, (b) Design drawing

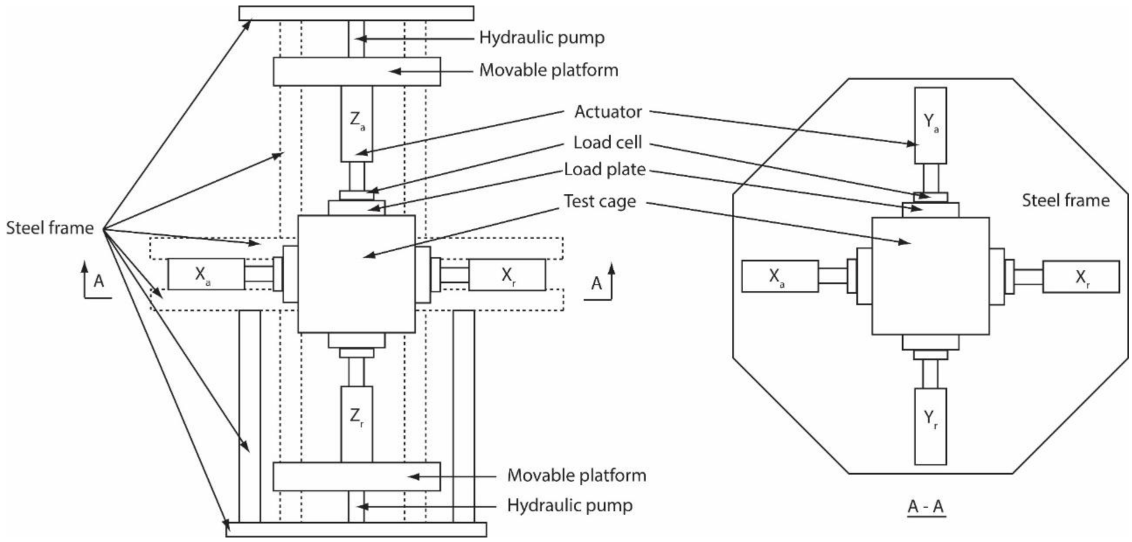


Figure 2. Schematic of the GeoTT (Left: side view, Right Birdseye view – not to scale)

2.2 Test cage

A bespoke steel test cage was developed to confine the large granular particles during testing. The outer skeleton had dimensions, 560mm x 560mm x 560mm, and 6 hollow sides. Each of these sides housed 6 separate and independently movable walls that allowed the sample to change volume during testing (Figure 3). Each wall had a maximum stroke of 60mm to prevent each wall colliding. The skeleton had protruding protective stops (not shown) to prevent sample egress in the event of excessive wall contraction (Figure 3a). These stops were linked to the control software and the test would automatically halt if this condition was reached. Further, to prevent small granular particles from exiting the sample via the skeleton-wall clearance, the inner test cage was encased using a thin plastic membrane (see Figure 4).

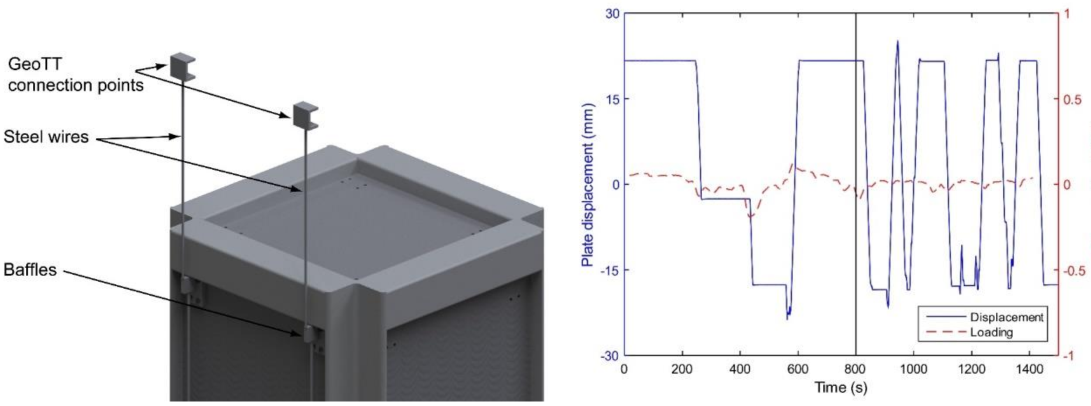
The true triaxial tests also depended upon the cage wall movement being independent from the cage skeleton. If friction was encountered at this location then the metal-on-metal contact could have introduced testing errors. During initial rig development it was found that this friction risk was greatest in the vertical plane, due to potential sag of the horizontally orientated rams. Therefore a suspension system was developed to support the self-weight of the steel walls and load cells, thus counteracting the downward vertical force on the horizontal rams (Figure 5a). This was implemented by connecting the cage walls to the upper GeoTT frame via tuneable-length steel wires.

To illustrate the performance of the suspension system, Figure 5b shows a test performed during GeoTT commissioning, where the position of a lateral cage wall was cycled between the inside and outside of the cage skeleton. At time prior to 800s (shown by the black line), the suspension system was not engaged, however after 800s it was engaged. At all data points the measured horizontal force was recorded to quantify the potential horizontal resistance due to friction. In absence of the suspension system, the force varied from -0.20kN to 0.12kN depending upon position, while when present the force varied from -0.09kN to 0.04kN. Therefore, when the suspension system was engaged, the friction between walls and cage skeleton was significantly reduced. Accordingly, the suspension system was used for all tests presented in this paper.

(a) (b)

Figure 3. GeoTT testing cage: (a) walls contracted, (b) walls compressed

Figure 4. Inner cage with plastic sheet



(a) (b)

Figure 5. Suspension system details: (a) suspension design, (b) wall-skeleton friction

3 Testing methodology

Two sets of tests were undertaken. First, monotonic axial loading tests were performed for the purpose of investigating the Poisson's ratio and modulus of ballast. Next, a combined hydrostatic loading and unloading test was performed to further investigate the cross-anisotropic behaviour of ballast. Both tests following independent loading plans.

3.1 Sample preparation

The particle size distribution (PSD) of the railway ballast material was characterised in accordance with BS EN 13450-2002 (BSI, 2002) / BS EN 13450-2013 (BSI, 2013), with all particles lying in the 20-63mm range and d_{50} =43mm (see Figure 6). The coefficient of uniformity C_u and coefficient of curvature C_c were determined as 1.36 and 1.009 respectively, indicating the ballast was classified as uniformly graded. The ballast aggregate was also washed and dried in accordance with EN 13450-2002 (BSI, 2002) / BS EN 13450-2013 (BSI, 2013) and BS EN 933-1 (BSI, 2005). After the ballast was prepared, it was poured into the test cage (500mm x 500mm x 500m) in 5 stages, and each layer was compacted for exactly 10 minutes using a vibrating Kango tool to achieve a specimen density of 1,300kg/m^3.

Although only particle size distribution tests were used to characterise the ballast, it was sourced from the same Network Rail approved quarry as the ballast used by Kwan, (2006). Therefore the properties were likely to have been similar to those found in other UK ballast research works [e.g. LAA index ≤ 20 (BSI, 2010), MDE index ≤ 7 (BSI, 2011), ACV ≤ 22% (BSI, 1990), Flakiness index ≤ 35 (BSI, 2012), Particle length ≤ 4 (BSI, 1996)].

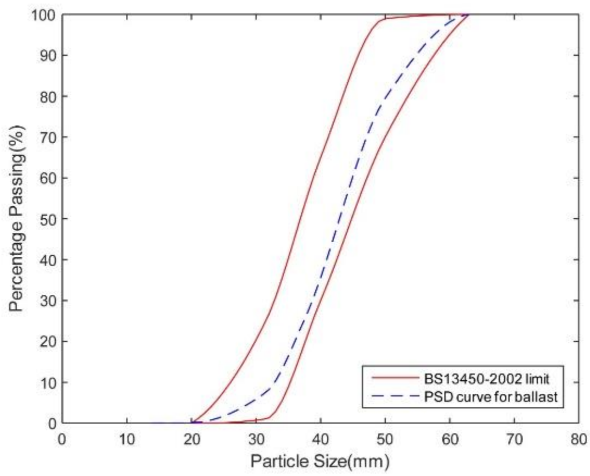


Figure 6. Ballast particle size distribution curve

3.2 Test 1: monotonic Axial Loading

The six rams (X_a, X_r, Y_a, Y_r, Z_a, Z_r) were used to apply static compressive stresses towards ballast samples as shown in Figure 7. Subscripts 'r' and 'a' are used to differentiate between the 2 different rams in each Cartesian plane. The axial load direction was varied between the X, Y and Z planes, and had a maximum value of 500kPa. Three different confining stresses (30, 60, 75kPa) were applied to the four specimen faces and each test (e.g. M1-9) was repeated three times, resulting in a total of 27 test results. The selection of these three confining stressed was based on the research from Indraratna et al. (2009), where three degradation zones were identified with confining stress: less than 30kPa, 30-75kPa and larger than 75kPa. For each test, the following procedure, also summarized in Table 4, was used:

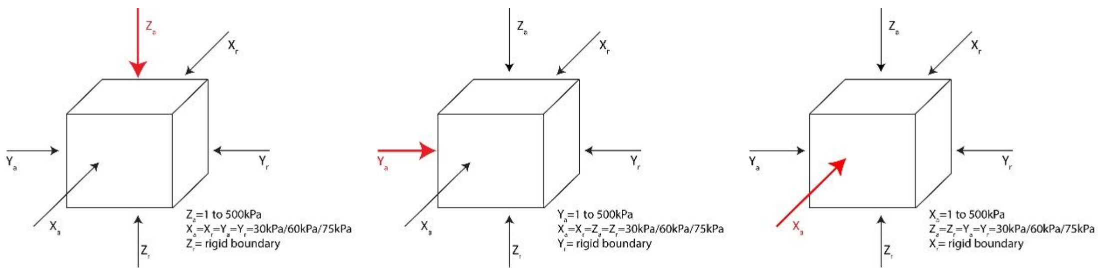
a) A constant confining stress (either 30kPa/60kPa/75kPa) was applied to the ballast sample in the 2 directions that were not the primary loading direction;
b) The position of the loading plate was recorded to determine the initial length of the sample (L);
c) An axial stress was applied in the primary loading direction and increased

monotonically at a rate of 62.5kPa per minute, from an initial value of 6.25kPa. The opposite ram maintained a fixed position, thus creating a rigid boundary;

d) When the axial stress reached 500kPa, it was held constant for 5 minutes;
e) The axial stress was decreased at a rate of 62.56kPa per 10 seconds until reaching a magnitude of 6.25kPa;
f) X, Y and Z displacements were recorded throughout steps a-e;
g) Steps a-f were repeated three times on the sample to ensure repeatability and consistency of results;
h) Steps a-g were repeated for the remaining axial loading directions; and finally,
i) The confining stress was increased (3 values tested: 30kPa/60kPa/75kPa) and steps a-h repeated.

Table 4. Monotonic axial test procedure

Test stage	Confining stress (kPa)	Confining direction	Axial load direction	Rigid boundary
M1	30	X and Y	Za	Zr
M2	30	X and Z	Ya	Yr
M3	30	Y and Z	Xa	Xr
M4	60	X and Y	Za	Zr
M5	60	X and Z	Ya	Yr
M6	60	Y and Z	Xa	Xr
M7	75	X and Y	Z_a	Z_r
M8	75	X and Z	Y_a	Y_r
M9	75	Y and Z	X_a	X_r



(a) (b) (c)

Figure 7. Axial load directions: (a) Z_a loading, (b) Y_a loading, (c) X_a loading

3.3 Test 2: combined Hydrostatic Loading and Unloading

In addition to the monotonic axial loading tests, a combined hydrostatic loading and unloading test was also performed to investigate the unloading response of ballast. Rather than using the same procedure as for the previous monotonic tests (Table 4), the hydrostatic compression test procedure outlined by Desai, Siriwardane and Janardhanam (1983) was used and is summarized in Table 5:

a) A hydrostatic confining stress of 34.5kPa was applied in all 3 directions
b) The axial stress was increased in increments of 34.5kPa, from the confining stress (34.5kPa) to 172kPa. At each increment, the deformations were measured after they stabilized.
c) The axial stress was reduced from 172kPa to 34.5kPa in 34.5kPa increments
d) The axial stress was increased in increments of 34.5kPa, from the confining stress (34.5kPa) to 345kPa. At each increment, the deformations were measured after they stabilized.
e) The axial stress was reduced from 345kPa to 34.5kPa, in 34.5kPa increments
f) The axial stress was increased in increments of 34.5kPa, from the confining stress (34.5kPa) to 517kPa. At each increment, the deformations were measured after they stabilized.
g) The axial stress was reduced from 517kPa to 34.5kPa, in 34.5kPa increments;
h) The test was repeated for the remaining two Cartesian planes.

Table 5. Hydrostatic testing procedure

Test stage	Confining stress (kPa)	Deviator stress (kPa)	Confining direction	Axial load direction	Rigid boundary
H1	34.5	137.5	X and Y	Z_a	Z_r
H2	34.5	310.5	X and Z	Y_a	Y_r
H3	34.5	482.5	Y and Z	X_a	X_r

3.4 Interpretation of test results

When a uniaxial compressive force is applied to a cubic or cuboidal test specimen, it contracts in the axial direction and expands in the remaining two perpendicular/transverse directions (Figure 8). Assuming the axial stress is in the Z direction, the resulting recoverable horizontal strains are in the X and Y directions (see Figure 9). For this case, Equations (1), (2) and (3) give the calculation for the axial recoverable strain in the Z direction and the horizontal (or transverse) recoverable strains in X and Y directions, respectively. Then, the magnitude of average horizontal or transverse strain is calculated using Equations (4):

$$\varepsilon_{axial} = \frac{\delta_z}{L}\text{, (positive for axial compression)} \tag{1}$$

$$\varepsilon_{trans_x} = \frac{\delta_x}{L}\text{, (negative for axial compression)} \tag{2}$$

$$\varepsilon_{trans_y} = \frac{\delta_y}{L}\text{, (negative for axial compression)} \tag{3}$$

$$\varepsilon_{trans} = \frac{\varepsilon_{trans_x} + \varepsilon_{trans_y}}{2} \tag{4}$$

Where,

L is the initial length of the ballast sample

δ_x is the recoverable displacement of ballast sample in X direction

δ_y is the recoverable displacement of ballast sample in Y direction

δ_z is the recoverable displacement of ballast sample in Z direction

Thus, Poisson's ratio is calculated as:

$$\nu = -\frac{\varepsilon_{trans}}{\varepsilon_{axial}} \qquad (5)$$

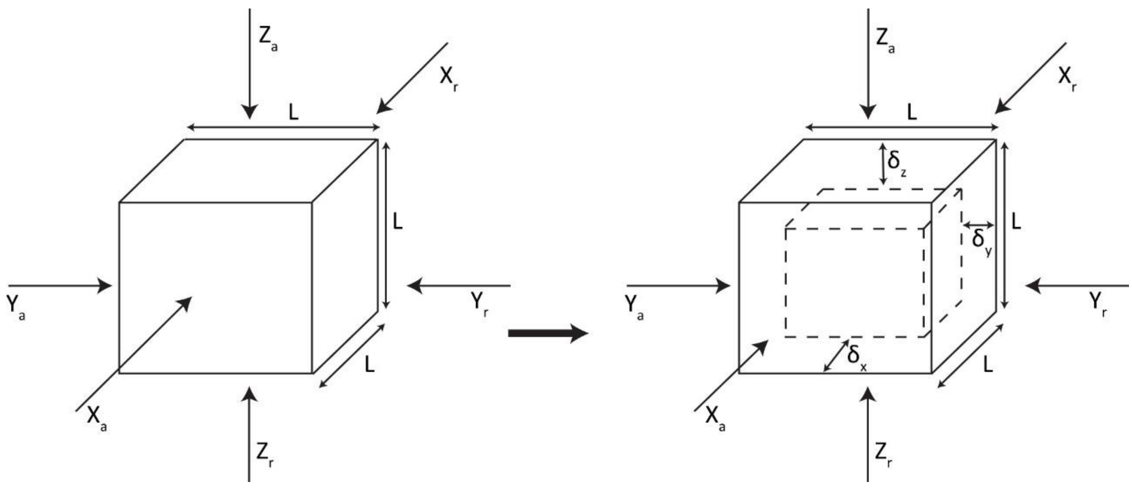
and the modulus is calculated as:

$$Modulus = \frac{\sigma_{axial\_time}}{\varepsilon_{axial\_time}} \qquad (6)$$

Where,

σ_{axial_time} is the axial stress time history

ε_{axial_time} is the axial strain time history



(a) (b)

Figure 8. Poisson's Ratio calculation: (a) un-deformed specimen, (b) deformed specimen in dashed line

4 Test results and discussion

4.1 Test 1 results

4.1.1 Poisson's ratio

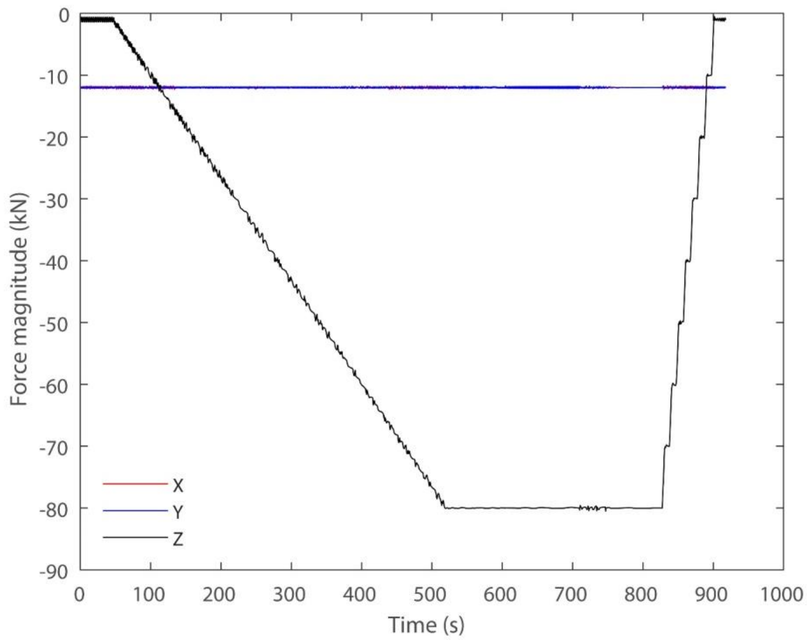
For each confining stress and axial load pair, 3 tests were performed. As an example, Figure 9 shows the loading path and displacements in X, Y and Z directions for monotonic axial loading in the Z_a direction under a confining stress of 75kPa. The displacements were used for the calculation of strains, while only recoverable strains were used for the calculation of Poisson's ratio. It is seen that deformation did not return to zero after unloading, thus indicating plastic deformation or displacement. This plastic deformation evinced that the further sample compaction was occurred in the vertical direction while the sample expanded in horizontal direction when the primary loading was applied in Z_a.

Since 3 repeated tests were performed under the same confining stress and axial load direction, the mean value of Poisson's ratio was calculated to minimize the test error (e.g. the 3 Poisson' ratios in the Za direction from stage M1 with 30kPa confining were averaged, giving a mean Poisson's ratio of 0.31).

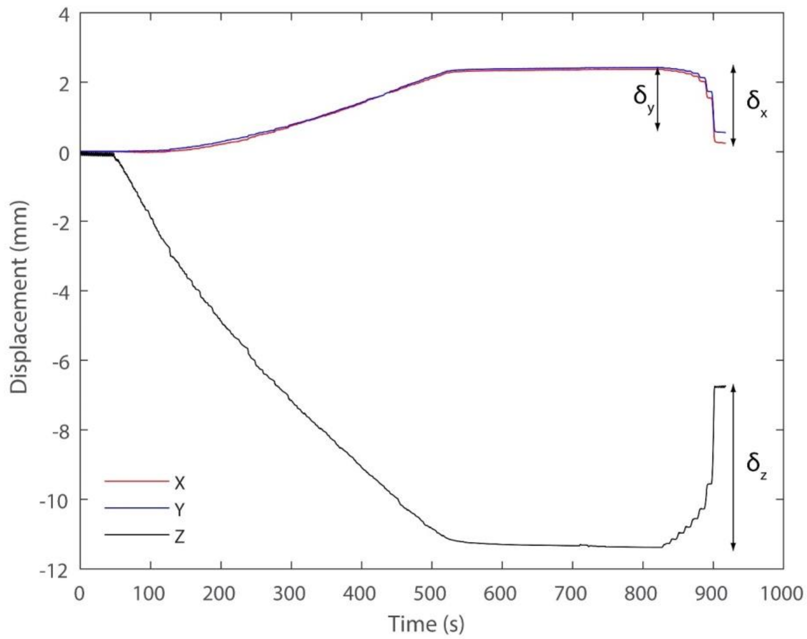
Figure 10 shows the relationship between confining stress and Poisson's ratio. It is seen that there was a distinct correlation, with Poisson's ratio decreasing with increased confining stress. The mean values in X, Y and Z directions were 0.28, 0.32 and 0.31 at a confining stress of 30kPa. The Poisson's ratio values reduced to 0.22, 0.26 and 0.23 for a confining stress of 60kPa and further reduced to 0.15, 0.19 and 0.18 for a confining stress of 75kPa (Table *6*). This is mainly due to that the increased confining stress reduced the displacement in the confining direction, resulting a reduced Poisson' ratio. This was consistent with the ballast triaxial tests performed by Indraratna et al. (1998), where the initial loading stage was used for the study between confining stress and Poisson's ratio in the vertical direction. It was found that Poisson's ratio decreased with increasing confining stress. Similarly, Aursudkij et al. (2009) carried out cyclic triaxial tests and found that the vertical Poisson's ratio decreased as confining stress increased from 30kPa to 60kPa under axial loading. The small discrepancies between the X and Y directions were likely because the monotonic test in the X direction was performed after the Y direction, resulting a lower Poisson' ratio in the X direction.

Table 6. Poisson's ratios for monotonic lading in X, Y and Z directions

Confining stress (kPa)	30			60			75		
Axial load direction	X	Y	Z	X	Y	Z	X	Y	Z
Mean value of Poisson's ratio	0.28	0.32	0.31	0.22	0.26	0.23	0.15	0.19	0.18



(a)



(b)

Figure 9. Monotonic testing results when loading in Z_a direction under a confining stress of 75kPa: (a) loading path, (b) displacements in X, Y and Z directions

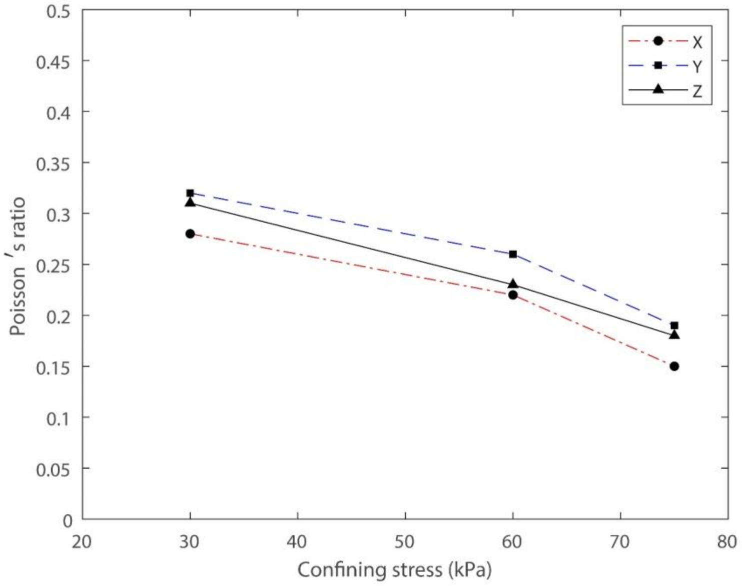


Figure 10. Relationship between Poisson's ratio and axial loading direction

4.1.2 Modulus

Figure 11 shows an example of the modulus from all repeated tests in X, Y and Z axial load directions under 75kPa confining stress. Correspondingly, Figure 12-Figure 14 show the mean modulus (average modulus across the 3 repeated tests) for confining stresses of 30, 60 and 75kPa respectively. Considering the Z direction at 30kPa, Figure 12 shows that the modulus decreased rapidly when the axial stress was low, reached a local minimum and then increased again steadily as axial stress was increased. This dilation resulted in the modulus at the end of the test (500kPa) being similar to the starting value. Regarding the X and Y directions, their responses were similar and had modulus significantly lower than the vertical direction (Z is on average 73% higher than X, and 66% higher than Y, when the axial stress is 500kPa).

Similar findings were obtained for confining stresses of 60kPa and 75kPa. Figure 13 shows the 60kPa case where Z was on average 66% higher than X and 76% higher than Y when the axial stress was 500kPa. Alternatively, Figure 14 shows the 75kPa case where Z was on average 46% higher than X and 54% higher than Y when the axial stress was 500kPa. The overall combined results are shown in Figure 15 where it can be seen that for all directions, the lower confining stress resulted in lower modulus.

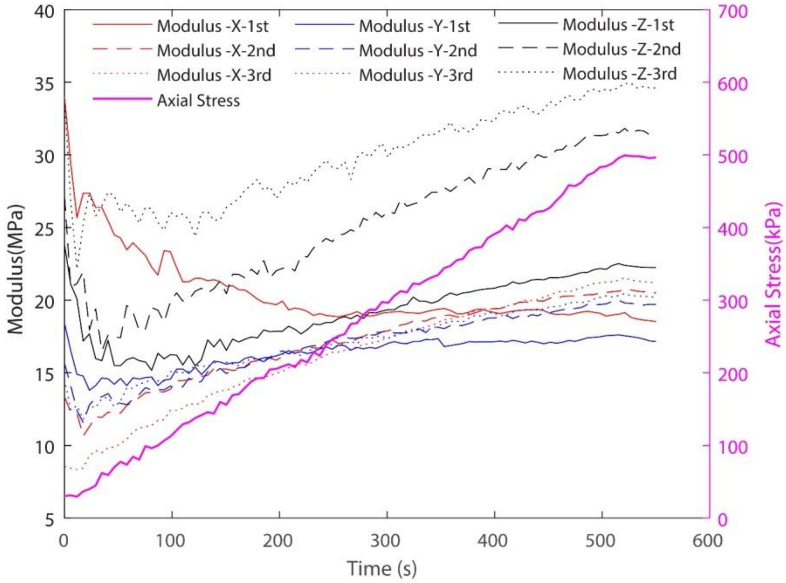


Figure 11. Raw modulus in X, Y and Z axial load directions under 75kPa confining stress

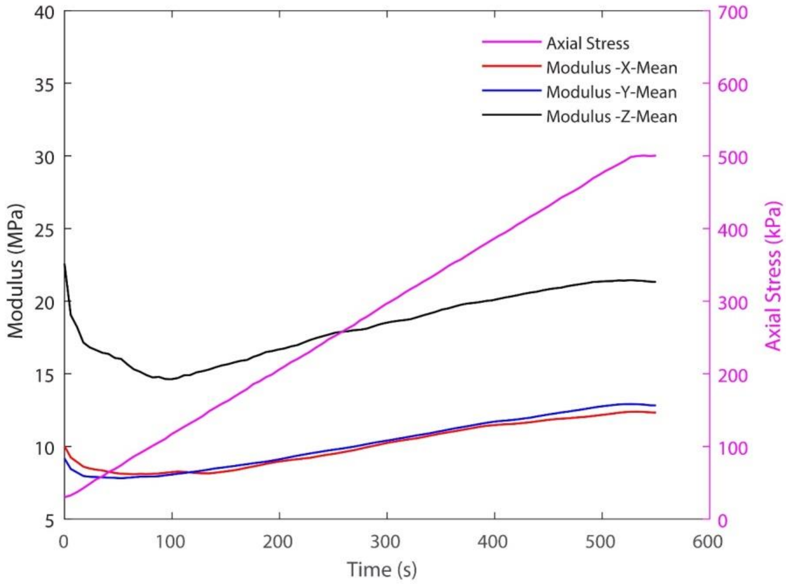


Figure 12. Mean modulus in X, Y and Z axial load directions under 30kPa confining stress

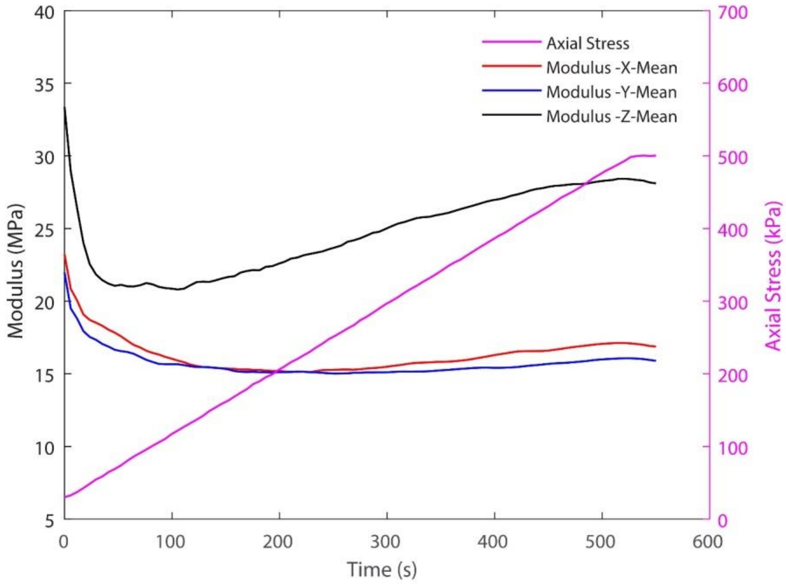


Figure 13. Mean modulus in X, Y and Z axial load directions under 60kPa confining stress

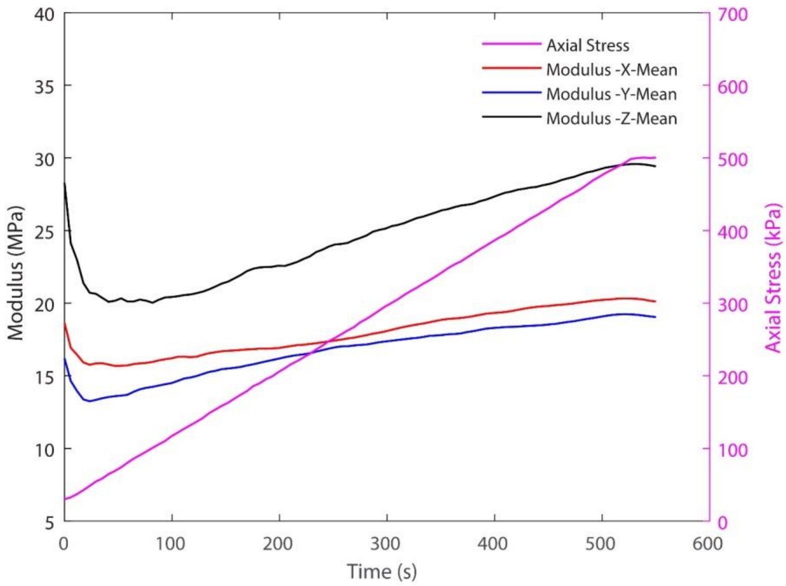


Figure 14. Mean modulus in X, Y and Z axial load directions under 75kPa confining stress

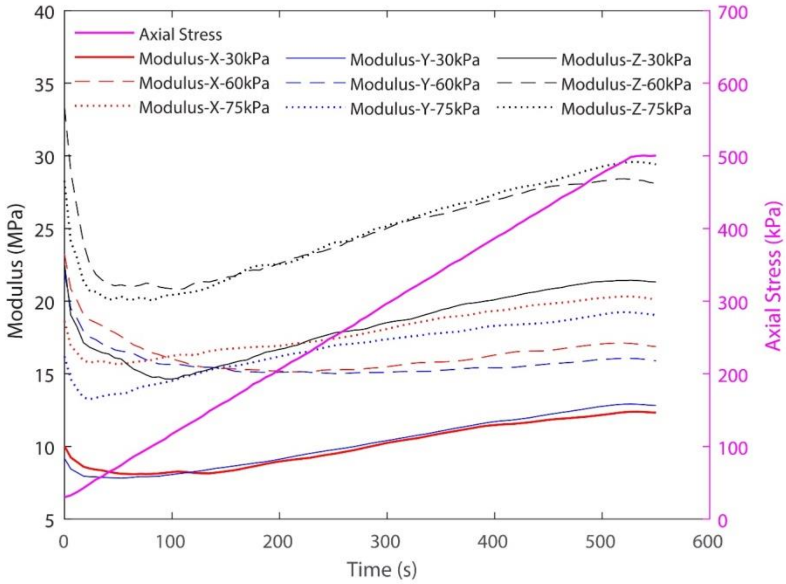


Figure 15. Mean modulus in X, Y and Z axial load directions under varying confining stress

4.2 Test 2 results

Figure 16 shows the three loading and unloading cycles of the test sample. Permanent deformation was clearly recorded after the 3 cycles in X, Y, and Z directions, and reached maximums of 172kPa, 345kPa and 517kPa, respectively. The horizontal directions (X and Y) exhibited approximately double the permanent deformation (3%) compared to the vertical direction (1.5%), thus demonstrating the typical cross-anisotropic behavior of ballast. Based on recoverable strains after each unloading stage, the relationship between sample modulus and deviator stress is presented in Figure 17. It shows that sample modulus in the X and Y directions increased from 640 MPa and 600 MPa, to 800 MPa and 700MPa respectively, when the deviator stress was 310.5 kPa. However, the sample modulus in the Z direction increased from 900 MPa to 1000 MPa when deviator stress reached 310.5 kPa, and then remained constant. This change of modulus in each direction again indicated the cross-anisotropic behaviour of the ballast sample, while this cross-anisotropic behaviour was similar in X and Y direction in terms of unloading response and sample modulus. However, it should be noted that the responses show discrepancies with traditional nonlinear soil behaviour.

Upon inspection after testing, ballast breakage was noticed when removing the sample from the test cage. Also, ballast aggregate particle cracking was heard during testing.

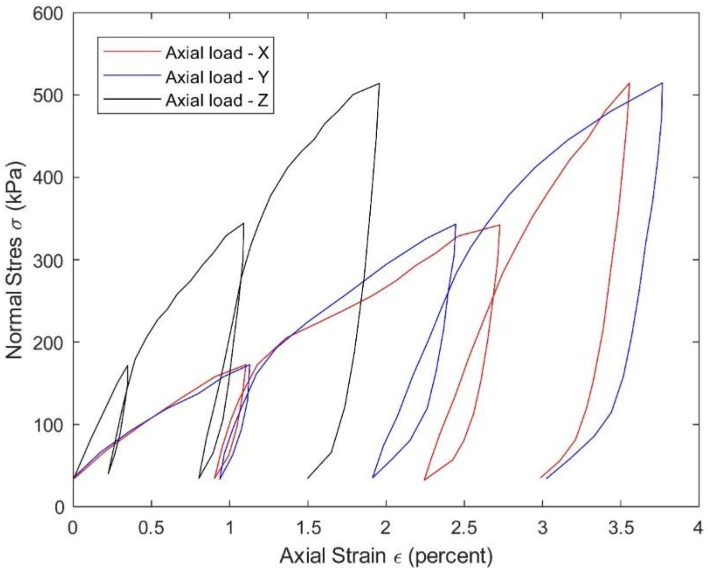


Figure 16. Unloading response in all three axial load directions

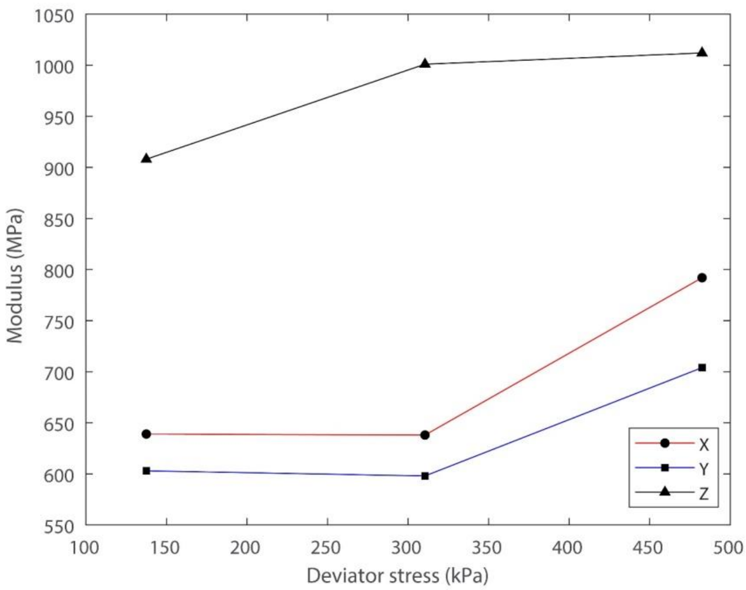


Figure 17. Relationship between modulus and deviator stress

5 Conclusions

Railway ballast typically behaves in an anisotropic manner, with greater stiffness in the vertical compaction direction. This is important to quantify for a better understanding of the field mechanical behaviour, and for the modelling of the dynamic response behaviour. Therefore, this paper presented results for a railway ballast aggregate material tested under true-triaxial conditions. A novel, large, true-triaxial apparatus and its accompanying testing cage were successfully developed in the laboratory. The true triaxial device utilised six hydraulic actuators to ensure a uniform stress distribution across the test sample. Three confining stresses (30kPa, 60kPa and 75kPa) were used to investigate Poisson's ratio, modulus and loading-unloading characteristics. Anisotropic behaviour was clearly observed for the ballast aggregate material; the horizontal response as obtained from horizontal and transverse strain measurements varied when compared to the strain values measured in the vertical direction. Both Poisson's ratio and modulus were sensitive to the applied confining stresses.

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