

Full Scale Testing of Ground Remediation Options for Residential Property Repair following the Canterbury Earthquakes

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ABSTRACT: A series of full scale tests have been undertaken to assess the performance of ground strengthening methods to improve seismic performance of liquefiable soils in the Christchurch area. The tests used sequences of explosive charges to simulate seismic shaking at levels representative of SLS and ULS events and induced liquefaction and expulsion of sand. Monitoring included measurement of ground motion, pore pressure development and settlements.

The results have determined that the treatment of the upper crust by densification or cement stabilisation is an effective method of reducing settlements and preventing surface expulsion of liquefiable soil. Other options including deep soil mixing and a perimeter curtain wall were less effective but achieved the proposed design objectives and also have application.

1 INTRODUCTION

Liquefaction during the recent large earthquakes in Christchurch, New Zealand, has caused significant damage to tens of thousands of houses. As the city rebuilds on land at risk of liquefaction damage, there is a need to develop simple, cost effective engineering measures which increases the resilience of new houses to limit losses and disruption from future earthquakes. A number of ground improvement methods have been trialled, with the purpose of developing recommended foundation solutions for reconstruction and repair of damaged houses in the Canterbury region.

The trial involved a series of full scale tests undertaken using four ground improvement options. The ground improvement options have been subject to simulated earthquakes generated by explosive charges. The tests have provided useful information for the development of foundation solutions for some of the areas of Christchurch that have been identified as being at a relatively high risk of moderate to severe liquefaction damage in the future. These areas have been classified as Technical Category 3 (TC3) by the Canterbury Earthquake Recovery Authority (CERA).

The foundation systems that were tested include densification of a crust layer, cement stabilisation of a crust layer, construction of a grid of deep soil mix columns, and a perimeter wall of contiguous piles. The test areas were formed around a central core in which a sequence of charges were detonated to generate shaking motion that was representative of the proposed serviceability limit state (SLS) and ultimate limit state (ULS) design earthquake events for Christchurch.

2 SITE LOCATION AND GEOLOGY

The trial was undertaken at Queen Elizabeth II Athletics Stadium, New Brighton, Christchurch, which has suffered significant liquefaction induced land and building damage during the recent earthquakes. The ground conditions comprised loose fine to medium sands to 6.5m depth, underlain by medium dense and dense sands which extend to over 20 m depth, see Figure 1. Groundwater was at 1 m depth.

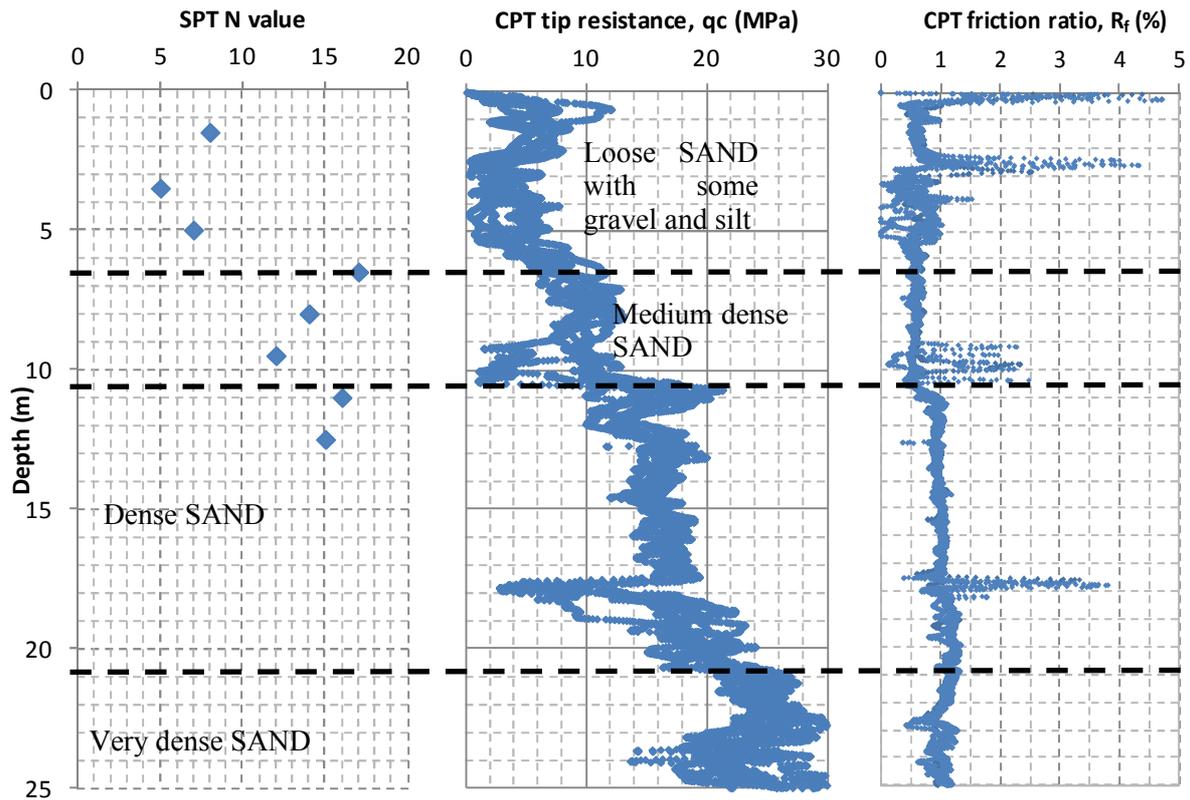


Figure 1 – Summary of site investigation results

Testing of the soils in the trial area included a borehole with SPT tests, core sampling and laboratory testing, 6 dynamic probe (heavy) testing and 5 Cone Penetrometers (CPT). Analyses were undertaken of the liquefaction risk for the site, and to assess liquefaction induced settlements using the method described by Moss et al (2006). These indicate the loose sands in the upper 6.5m are expected to liquefy in an SLS event while, in an ULS event, liquefaction of both the loose and medium dense sand layers to 10 m depth is expected, in addition to thin layers below 10m depth.

Associated liquefaction induced settlements were calculated at about 160-240 mm and 350-410 mm for the SLS and ULS events respectively. It is noted, however, experience in Christchurch indicates that this method tends to overestimate settlements unless they are associated with surface expulsion of materials.

3 TRIAL DESIGN

The concept developed for the trial comprised a ring divided into 5 segments of different ground improvement options, around a central core in which a series of charges would be detonated to simulate earthquake shaking. This arrangement subjected each segment to the same level of shaking, to enable a direct comparison of performance. Concrete blocks were used to simulate foundation loads. The general arrangement of the trial layout is shown on Figure 2.

3.1 Ground improvement

The ground improvement options selected are described as follows:

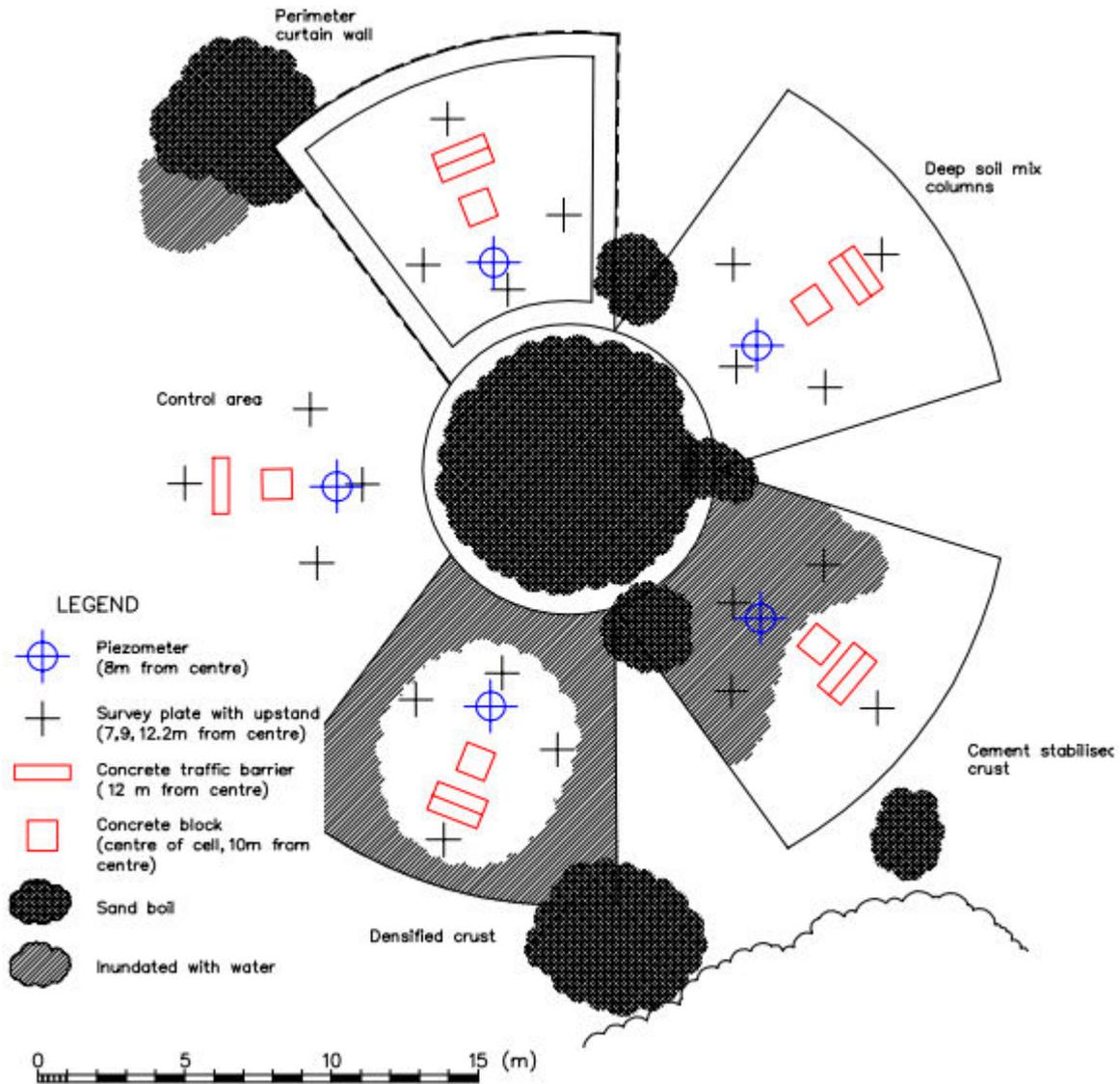


Figure 2 – Ground improvement trial layout, showing extent of ground improvement zones, instrumentation layout and observations following the second trial.

3.1.1 Densified Crust of Natural Soils (DC)

A 2 m layer of soil was excavated and recompactd in 200 mm layers to engineered specification to achieve approx 100% standard optimum density. The material achieved an average Scala target strength of about 5 blows/100 mm. The objective of this segment was to assess the performance of a dense surface crust layer.

3.1.2 Cement Stabilised Crust (SC)

A 2 m layer of soil was excavated, blended with 3% by weight of cement and replaced in 200 mm layers. The material was lightly compacted to achieve the natural density of the soil to represent insitu mixing of the cement with a panel mixer.

3.1.3 Deep Soil Mix columns

A Soilmec SR40 turbojet mixer was used to construct approximately 800 mm diameter columns on a 2 m grid pattern. The columns were targeted to achieve a depth of about 8 m. The cement dose rate used was about 12-15% to target 2-3 MPa strength and 400 MPa Youngs Modulus at 14 days.

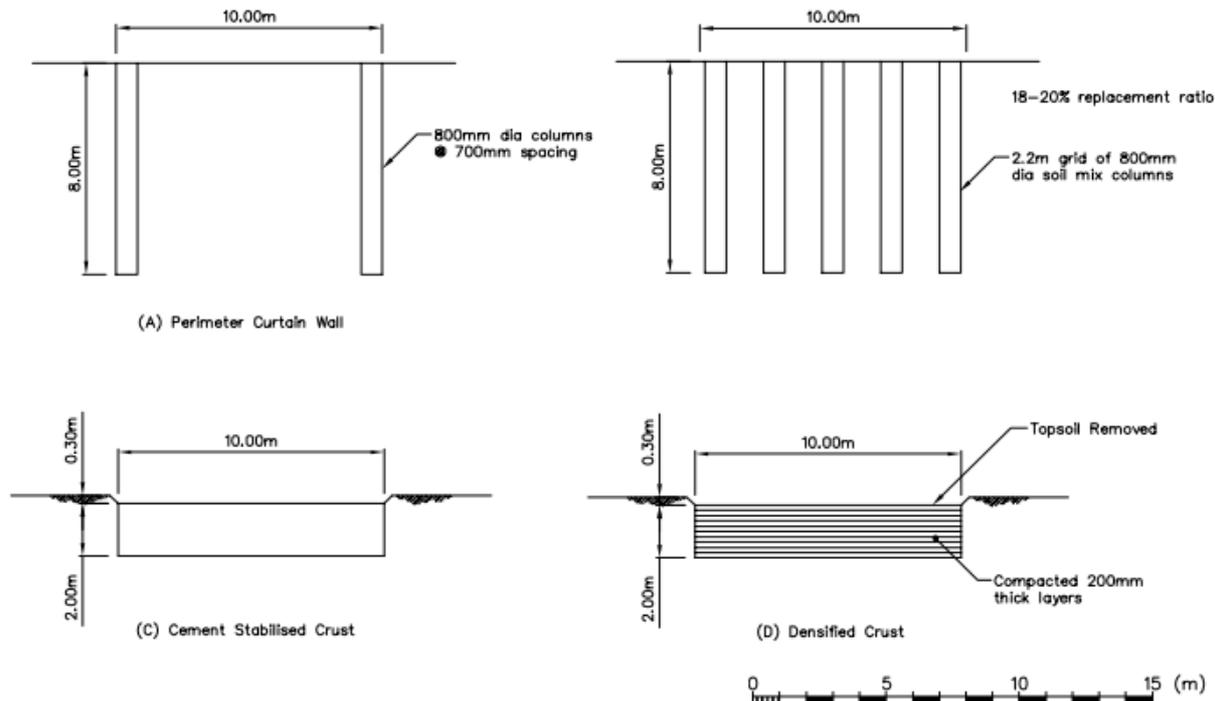


Figure 3 – Ground improvement details used in the trial

3.1.4 Perimeter Curtain Wall

A Soilmec SR40 turbojet rig was used to form the series of 800 mm diameter secant deep soil mix columns. The objective was to utilise the stiffness of the cellular structure to reduce shear strains within the confined zone thereby limiting the rate of excess pore pressure development. A theoretical analysis showing the effectiveness of a similar concept was provided by Bowen (2007).

3.2 Blast Designs

Explosives have been successfully used to simulate earthquake events in a number of published studies (Struder and Koh, 1980; Qasimi, Charlie and Woeller, 2005; Ashford et al. 2009; Eller 2010). In general, these studies indicate a minimum peak particle velocity (ppV) of 100 mm/s is necessary to achieve liquefaction in susceptible soils using single charges while for multiple charges, a ppV of 80 mm/s is sufficient.

The design for the first trial targeted a peak particle velocity of about 100 mm/s at the centre of the segments and utilised 10 charges of 5 kg of Power Flag explosive set with 400 m sec delays. The explosives were loaded in 6 m deep PVC lined holes and stemmed with about 4 m of tamped sand. The first trial achieved shaking levels that were equivalent to the SLS level event.

The second trial targeted a shaking level equivalent to an ULS event, increasing the target ppV to 150-200 mm/s at the centre of the segments. A longer sequence of 30 charges was included, loaded in 15 holes of 12 m depth each with 2 decked charges of up to 8 kg MIC.

A further feature of the second (ULS) test was to detonate a series of 10 charges of 3 kg MIC loaded in holes drilled around the outside of the trial area. These were detonated immediately in advance of the main charges in the core, with 17 ms delays between the charges. This was designed to create a pressure ‘front’ for the pore pressure response to assist in reducing the gradient of pore pressure that results from the radial layout of the test.

3.3 Instrumentation and site monitoring

The testing undertaken to assess the performance of each of the segments included:

- Survey measuring points located on concrete ‘foundation’ blocks applying both 20 kPa and 12 kPa bearing stresses
- A grid of survey plates buried at 300 mm depth with upstands
- Triaxial geophones, buried at the centre of each segment (Nomis and Instanetel seismographs)
- Sealed piezometers located at 3 m depth in the centre of each segment (Solinst Model M100 levellogger – over pressure rating of 2 MPa, maximum recording rate 8Hz)
- Dynamic penetrometer test (heavy) before and at completion of each test.

4 TEST RESULTS

4.1 First Test

In the first test 10 charges of 5 kg were detonated in sequence with 400 msec delays. Peak particle velocities of 150-200 mm/s were recorded at the centre of each segment, with between 4 and 6 significant cycles of shaking. This corresponds to an inferred PGA of 0.3 – 0.65g. The attenuation relationships indicate a maximum of 400 mm/s at the inner radius and 80 mm/s at the outer radius of the ring. A typical record of the event is shown in Figure 4 Analysis of the results provides an estimated equivalent magnitude of shaking of an M5.5 – 6.3 event in close proximity of the site.

The piezometers recorded excess pore pressures of up to 2.7 m at the centre of the segments reducing to 0.7 m outside the ring. These correspond to an excess pore pressure ratio ranging 0.5 to 1.0. However, the pore pressures were not sustained, dissipating rapidly (within 30 seconds) after the shot sequence. The levels of pore pressure recorded were adequate to cause liquefaction at depth and some cracking of the surface was observed but they did not result in any surface expulsion of sand and water.

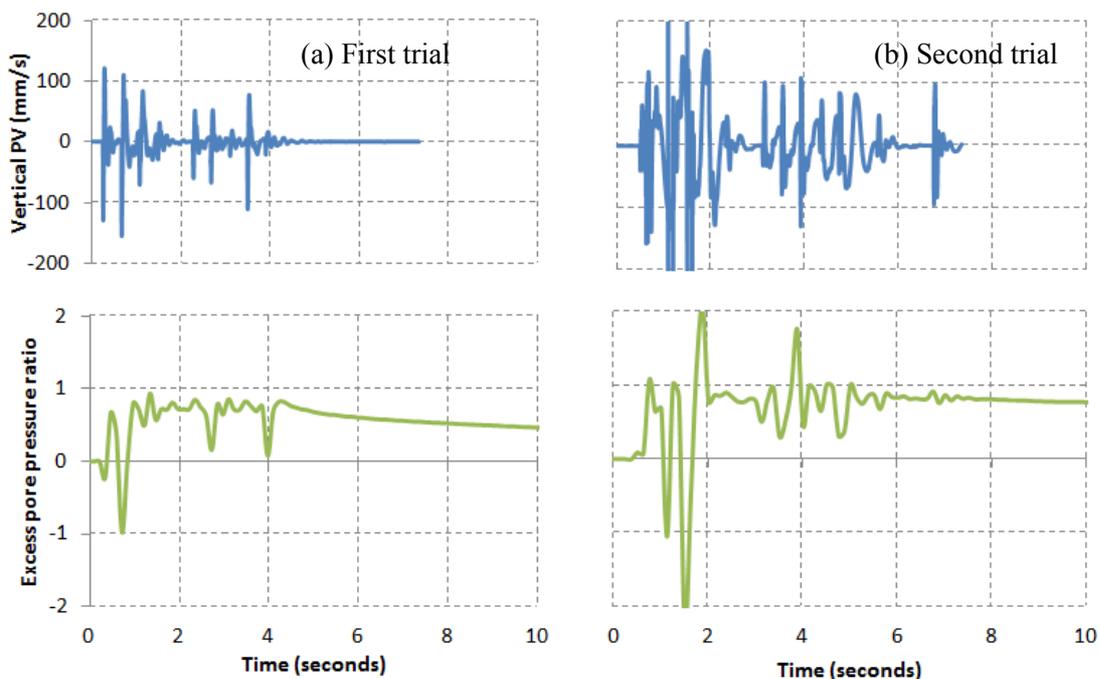


Figure 4 - Vibration monitoring results and pore water pressure response for deep soil mix segment

The survey measurements show that settlements were generally small, less than 20 mm for the concrete blocks. The settlement of the free surface of the segments was also generally less than 20 mm except for a local area near the inside edge of the control segment where settlements of 40-60 mm were recorded. There was an approximate 30% increase in settlements recorded between reading 1 hour after the blast and the following 24 hours.

4.2 Second Test

The second test included 10 charges of 3 kg detonated around the perimeter of the test area using 17msec delays in advance of 30 charges in 12 m deep holes in the core area. The inner and outer rings were loaded with charges of 8kg and 5.6kg respectively; two charges were ‘decked’ in each hole, one at 7.7m depth, the other at 12m depth.

Observation of the test showed no significant ejection of material during the blast sequence. Several water spouts were observed some 10-15 seconds after completion of the blasting, with water reaching an estimated height of about 5 m. Sand boils formed in about 10 locations within and around the test area, mostly originating near shot holes but also along the inside edge of the stabilised crust segment where the contractor had difficulty in maintaining a stable face. Expulsion of material continued for about 10 minutes after the blast. Ground cracking was observed along the inside edge of the perimeter curtain wall and deep soil mix column segments, although no sand boils formed in these areas.

The results show the maximum peak particle velocities recorded in the centre of the segments were close to or slightly exceeded the maximum range (250mm/s) of the seismographs. The attenuation relationships indicate the ppV’s ranged from 200mm/s at the outer edge and 600mm/s at the inner edge of the segments. This corresponds to accelerations of 0.3 to over 1.0 g respectively. A typical record of the shaking is shown in Figure 4.

The number of significant cycles is 10-13 which corresponds to a M7-7.5 event centred close to the site, and exceeds a ULS design event. The piezometers peaked about 2.5-3 seconds from the initial detonation i.e. about 5 to 6 cycles, see Figure 4. The peak excess pore pressure ranged between 1.0 to 2.0 with the higher figures being recorded in the control, deep soil mixing and perimeter curtain wall segments. These levels of excess pore pressure were maintained for a period of over 5 minutes and progressively dissipated over the following 40 hours. The densified crust segment sustained settlements of 40-60 mm while the settlements of the stabilised crust were generally less than 20mm.

Table 1 - Second Trial Settlements

Segment	Settlements (mm) - Range and (Average)				Comment
	Large Blocks	Small Blocks	Settlement Plates	Surface Grid	
Control	129-134 (127)	114-121 (119)	98-136 (118)	80-236 (130)	Depression in the left front corner. Sand boil outside segment.
Densified Crust (DC)	48-65 (52)	50-66 (59)	17-56 (33)	20-60 (50)	Surface intact. Sand boil outside segment.
Stabilised Crust (SC)	4-11 (9)	11-15 (12)	1-17 (9)	10-30 (20)	Sand boils outside along inside edge of segment
Deep Soil Mixing (DSM)	44-81 (68)	54-62 (58)	39-58 (50)	50-70 (60)	Sand boil inside core near edge of segment
Perimeter Curtain Wall (PCW)	84-120 (112)	96-107 (100)	83-95 (89)	50-80 (70)	No ejection within the segment but sand boils adjacent to the inner and outer walls of the segment

5 CONCLUSIONS

The trials have demonstrated that the use of explosive charges is an effective method of modelling earthquake events. This trial has enabled a comparison of a number of ground improvement methods for both the SLS and ULS design earthquake events. Key conclusions include:

- The most effective and likely lowest cost method of ground improvement is to construct a high strength capping layer. A recompacted 2m crust provided a reduction of shaking induced settlements by over 50% under an equivalent ULS event but a larger improvement of over 80% was achieved by cement stabilisation of a 2 m layer.
- The results indicate a relationship between increased strength of the surface crust and settlement of the underlying liquefied soil. This indicates this method might be effective regardless of the thickness of liquefiable soil.
- The use of deep soil mix columns was effective in reducing settlements by 50% but may have achieved higher results if the piles had extended deeper – the columns were 8m deep, liquefaction is estimated to have occurred over the upper 12m of the soil profile. The columns also ensured a uniform settlement of the site.
- The perimeter curtain wall was trialled as a method that could be applied around existing buildings. It was the least effective with a 30% average reduction. Again, the wall did not extend to the base of the liquefied layer.

The aftershocks of 23 December 2011 caused the trial area to liquefy again – at the time of writing, analysis work is being carried out to see if additional data from that event might provide further insights into the performance of the trial improvement methods.

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