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Dynamic Analysis of a Shallow-Founded Building in Christchurch during the Canterbury Earthquake Sequence

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ABSTRACT

The 2010-2011 Canterbury earthquake sequence had several earthquakes of moment magnitude (M_w) 5 or greater in a period of two years which affected Christchurch, New Zealand. The different earthquakes resulted in different levels of ground and building damage related to liquefaction. This paper presents the results of dynamic soil-structure interaction (SSI) analysis of a 7-story steel frame building founded on reinforced concrete shallow footings that suffered liquefaction related damage. The results of monotonic and cyclic triaxial tests performed on "undisturbed" samples retrieved from the site provided the basis for the soil modeling. Seismic site response analyses estimated that liquefaction would be triggered where it was expected from simplified empirical procedures and importantly, at locations consistent with field observations. The results of the SSI analyses that included the building compare well with the observed response after three different earthquakes.

Introduction

The estimation of liquefaction-induced settlements on structures is largely based on empirical procedures to estimate post-liquefaction, one-dimensional (1D) consolidation settlements in the free-field, neglecting the effects of the presence of a structure (e.g., Tokimatsu and Seed 1987). Recent research (e.g., Dashti et al. 2010a, Dashti and Bray 2013) concluded that the seismically induced displacements were often primarily controlled by shear deformations as a result of SSI-induced ratcheting and a bearing capacity type of failure as well as volumetric deformations resulting from partial drainage, sedimentation, post-liquefaction consolidation, and soil ejecta. The previously mentioned 1D empirical procedures can only capture the settlements related to volumetric strain (i.e., primarily post-liquefaction consolidation). For this reason, numerical analysis that can capture shear deformations becomes a useful tool to gain insight.

In this paper the results of the numerical analysis of a building that suffered different levels of damage in different events during the 2010-2011 Canterbury earthquake sequence are presented. The analysis was performed using the program FLAC 2D (Itasca, 2009) and the user-defined model PM4Sand (Boulanger and Ziotopoulou, 2012). The calibration of the model was performed by modelling monotonic and cyclic triaxial tests (CTX) of "undisturbed" samples retrieved from the site at different depths and by then modeling a 1D soil column and comparing the liquefaction response of the soil column with simplified empirically based liquefaction evaluation procedures. Finally, the building and the underlying soil were modelled and the displacements were compared to the observed displacements after the different earthquakes.

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Site Description

Building Description

The building, herein called FTG-7, is a 7-floor steel frame structure supported on reinforced concrete (RC) strip footings with dimensions 23.9 m height, 29.1 m width (in the EW direction) and 31.8 m length (in the NS direction) (Zupan, 2014). The configuration of the structural frame as well as the foundation system is different in the NS direction than in the EW direction as shown in Figure 1a. In the NS direction, the foundation consists of two perimeters RC strip footings having an embedment depth of 1.2 m, a height of 0.6 m, a width of 2.4 m, and a length of 29 m. The four interior RC strip footings have an embedment depth of 0.6 m, a height of 0.6 m, a width of 3.3 m and a length of 25 m. The distances between the centerlines of the NS aligned footings and columns are between 5.5 and 6.3 m. The interior footings are connected with each other and with the perimeter footings in the EW direction through 0.6 by 0.6 m RC tie beams. In the EW direction, the perimeter footings are also embedded 1.2 m in the ground. They have a height of 0.6 m, a width of 2.0 m and a length of 34 m. These two EW aligned footings are connected to the previously described NS aligned footings (perimeter and interiors) through 0.6 by 0.6 m tie beams. The distance between the centerlines of the EW aligned tie beams and columns are between 6.0 and 6.8 m.

The columns are wide-flange steel sections with its web aligned parallel to the NS direction (Zupan, 2014). The dimensions of the W sections depend on the floor of the building and on the location of the column in plan view. Primary beams (W section) connect columns in the NS direction, while secondary (smaller) beams connect columns and primary beams in the EW direction. The size of the beams depends on the floor of the building and whether is an interior or perimeter beam as shown in Figure 1b. The ground floor consist of 0.1 m thick unreinforced concrete slab and floors 2 through 7 consisted of 0.12 m thick RC slab over 0.75 mm galvanized steel decking. The building coordinates are S 43.5263, E 172.6384, and additional details can be found in Zupan (2014).

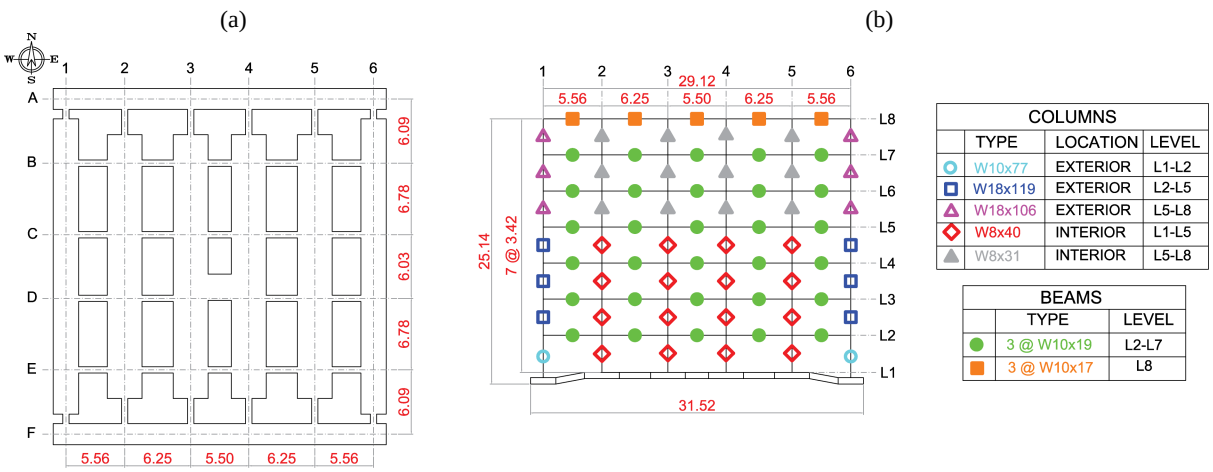

Figure 1. (a) Foundation plan view and (b) Structure's elevation view for lines B to E

Soil Conditions

The soil conditions in the site have been characterized using five CPTs, which were located near each corner of the building and in the center of the northern perimeter of the building

(Zupan, 2014). Additionally, three boreholes located near the southern corners of the building were performed. The site shows fairly uniform conditions. From the ground surface to 1-1.5 m depth there is a fill material with relative density (D_r) ≈ 60 to 80%. The next stratum goes to a depth of 7 to 8.5 m and is sandy silt – silty sand with variable fines content (FC) and soil behavior type index (I_c), generally between 2.0 – 2.5. The "clean sand" equivalent relative density for this deposit varies between 35 and 55%. The next stratum goes to a depth of around 14 to 16.5 m and consists of medium dense sand (I_c ≈ 1.5 and 2.0) with D_r ≈ 55 – 70%. After the medium dense sand, very dense sand (D_r ≈ 90%) is encountered and is usually within this stratum where the CPT reaches refusal. SASW data performed in the initial reconnaissance efforts detects the boundary of the Riccarton Gravel at a depth of 19 m. Some of the CPTs show a 1 meter thick layer of clayey soil between the Riccarton Gravel and the dense sand. The water table depth was about 2.0 meters throughout the earthquake sequence (Canterbury Geotechnical Database, 2013).

Constitutive Model: Description and Calibration

Calibration of constitutive model (PM4SAND) using laboratory data (monotonic and cyclic undrained triaxial tests)

The user defined model PM4SAND (Boulanger and Ziotopoulou, 2012) was used for the present study. The model follows the basic framework of the stress-ratio controlled, critical-state compatible, bounding surface plasticity model for sand presented by Dafalias and Manzari (2004). The model was calibrated using laboratory testing on thirteen "undisturbed" samples at different depths and within different soil strata that were tested in the laboratory either as an undrained monotonic or cyclic triaxial test. D_r was obtained through CPT correlations proposed by Idriss and Boulanger (2008), Kulhawy and Mayne (1990) and Jamilkowski (2001) weighted 0.5, 0.25, and 0.25, respectively. The soil's small strain shear modulus was estimated from the shear wave velocity (V_s), which was obtained from SASW data as well as from the CPT-V_s correlation proposed by McGann et al. (2014). More weight was given to SASW data for surficial layers whereas for deeper strata more weight was given to McGann's correlation.

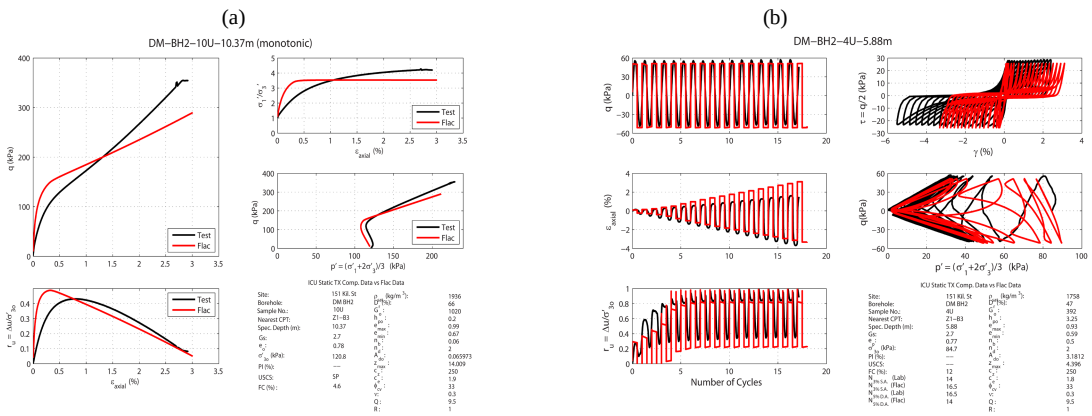

Figure 2. Comparison between laboratory test (black) and FLAC simulated test (red) for (a) Monotonic Triaxial Test (b) Cyclic Triaxial Test (CTX)

Parameters that define the critical state line (Q and R) and the maximum and minimum void ratio were obtained from the work of Rees (2010) on the effects of fines content on the critical state line and liquefaction triggering curve of Christchurch soils. The other parameters

were calibrated to match the monotonic or cyclic test data. Figure 2 shows representative results from the laboratory test and numerical model for a monotonic and cyclic test. It can be seen in the monotonic test how the stress-strain and stress path are well captured by the numerical model. In the cyclic triaxial test, the general trends in terms of pore water pressure generation and axial strains are also well captured, although the stress path proved difficult to match precisely. Figure 3 shows the results of the liquefaction triggering curve (CSR vs Number of cycles to get 5% double amplitude strain). The numerical model captures well the observed liquefaction response in the laboratory.

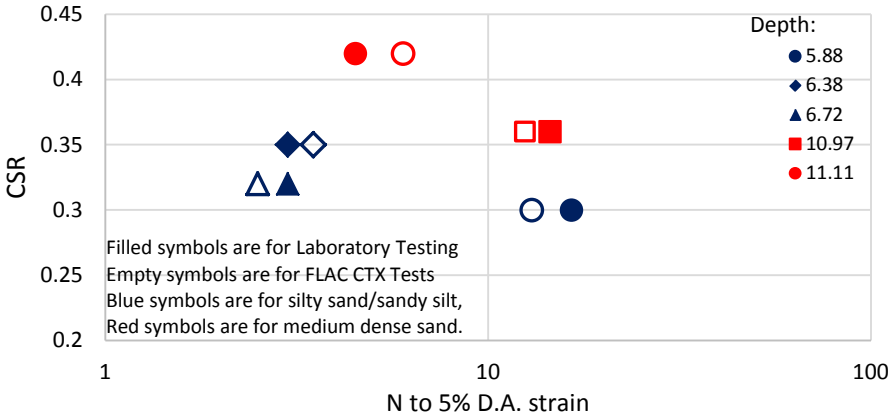

Figure 3. Comparison of number of cycles to get 5% double amplitude strain between laboratory test and FLAC simulated test.

1D Site Response Analysis

Following the calibration of each test for the site, a soil column was modelled and its seismic response was compared to the expected response from the simplified empirical liquefaction evaluation procedures as well as the calculated response spectra at the surface compared to the recorded response spectra in nearby strong ground motion stations where similar liquefaction related damage was observed. Three different events from the Canterbury earthquake sequence (CES) were analyzed: a) M_w = 7.1 on September 4th 2010 (Darfield event), b) M_w = 6.2 on February 22nd 2011 (Christchurch event), and c) M_w = 6.0 on June 13th 2011. The input ground motions were obtained through a SHAKE deconvolution process of recorded ground motions to the Riccarton Gravel as described in Markham et al. (2014) and then scaled based on Bradley (2013) ground motion prediction equation (GMPE) to account for the differences in source-to-site distance (R_{rup}) and shear wave velocity in the upper 30 meters (V_{s30}). Figure 4 shows the input properties and parameters versus depth.

Figures 5, 6, and 7 show the results for the Darfield, Christchurch, and June 13th event, respectively. In each figure the peak ground acceleration (PGA), peak ground velocity (PGV), maximum pore pressure ratio ($R_{u,max}$) and equivalent shear stress ratio (CSR_{eq}) profiles are compared with the factor-of-safety against liquefaction triggering (FS) estimated using the Boulanger and Idriss (2014) simplified CPT-based liquefaction evaluation procedure. Also, the base and surface response spectra at 5% damping are shown together with the recorded 5% damping spectra at the CCCC station, which is also located in the Christchurch Business District (CBD) and showed similar liquefaction response during the earthquakes as the site in study. For the 13th June event, the estimated surface spectrum was compared to the recorded at the REHS station, because the CCCC station did not record the 13th June event motion.

In general, the analytical results are reasonable. Liquefaction, defined in terms of maximum pore pressure ratio, is triggered where lowest FS were calculated. However, FS below unity did not always mean liquefaction was triggered. As an example, the simplified procedure predicts liquefaction triggering (FS < 1) from 2 to 7.25 m and from 9 to 14 m for the Darfield event (Figure 5). However, numerical analysis shows high pore pressures (>50%) only from 3.75 to 7.25 m.

The results of the numerical analysis are in agreement with field performance observations, which showed "*…there was minor surficial evidence of liquefaction in this area following the 4 SEP 2010 earthquake, severe and extensive liquefaction during the 22 FEB 11 earthquake, and moderate liquefaction during the 13 JUN 11 earthquake…*" (Zupan, 2014). For the Christchurch event (Figure 6) massive liquefaction is predicted, but the calculated response spectra is underestimated compared to the recorded motion at the CCCC station. A possible reason for this is that the constitutive model does not predict dilation spikes as high as those observed in the ground motion recordings. For the 13 JUN 2011 event a response only slightly less than the Christchurch event was calculated, even though the intensity of the ground motion was less.

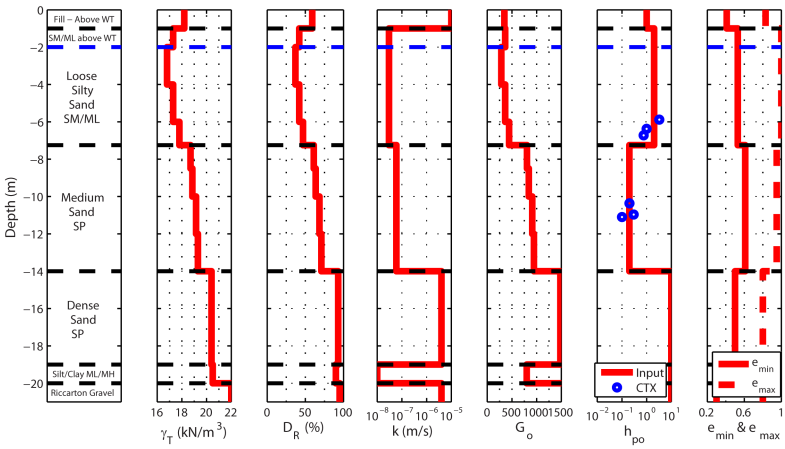

Figure 4. Input properties and parameters for the 1D seismic site response analysis.

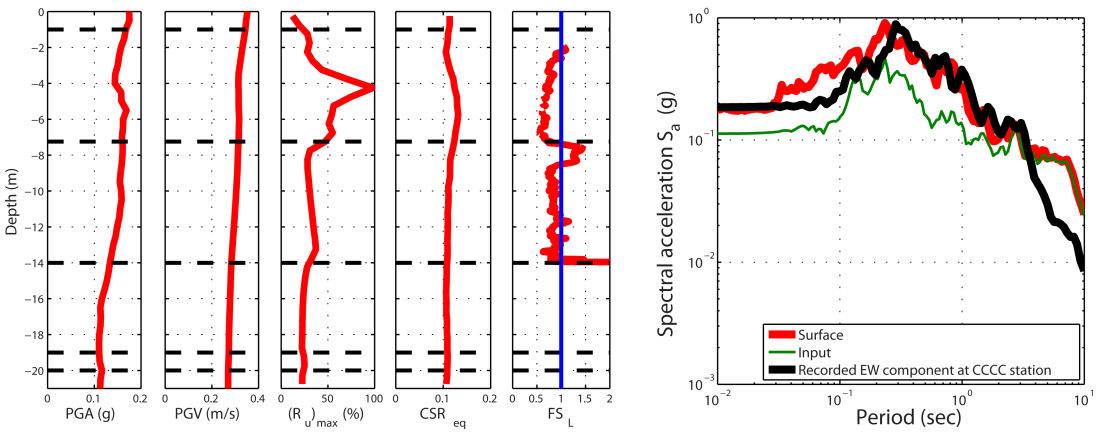

Figure 5. Results for a 1D column analysis in the EW component for the Darfield event.

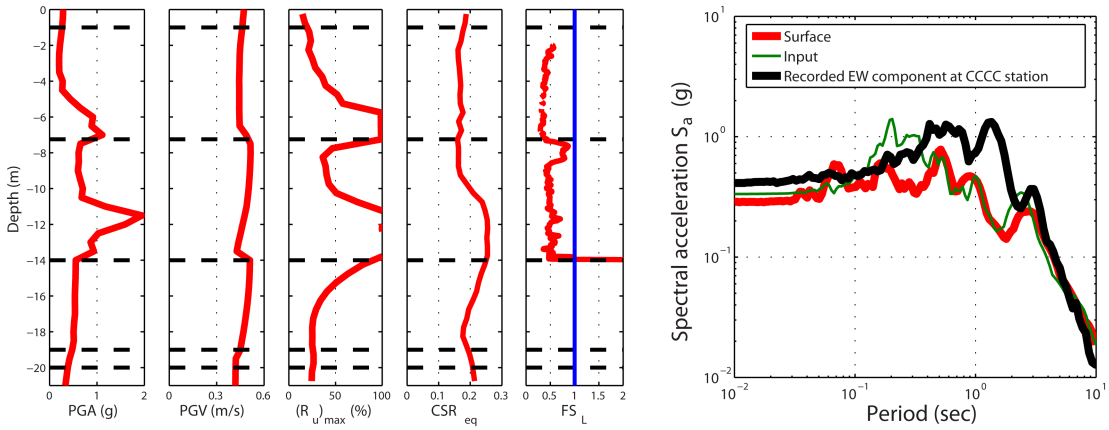

Figure 6. Results for a 1D column analysis in the EW component for the Christchurch event.

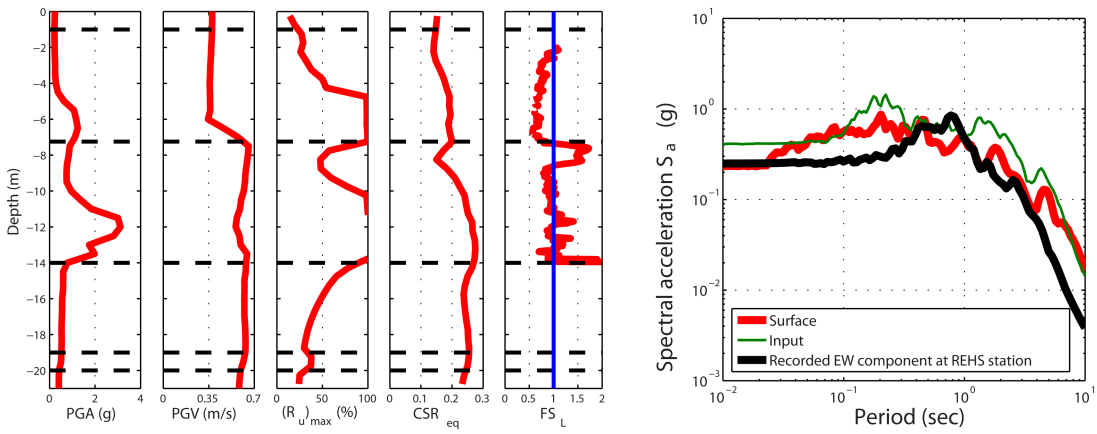

Figure 7. Results for a 1D column analysis in the EW component for the JUN-13-11 event.

Building Model and Response

Two typical frames oriented in the EW direction were modeled in the commercial software FLAC-2D (Itasca, 2009). The first frame corresponds to an interior frame (lines B to E in Figure 1a), and the other to a perimeter frame (lines A and F in Figure 1a). The frame elements (beams) were modeled using linear-elastic elements. The elasticity modulus and unit weight of concrete used for the analyses were 2.35×10^7 kPa and 24 kN/m^3, respectively. The elasticity modulus and unit weight of steel used for the analyses were 2.0×10^8 kPa and 77 kN/m^3, respectively. In elements where the stiffness of the system came from different materials (steel and concrete), an equivalent steel section was estimated such that the actual stiffness and weight of the system was obtained for the analysis. The area and second moment of inertia of the W sections were obtained directly from the AISC Database (AISC, 2014). The 3D properties of the building were considered by introducing a typical frame spacing of 6.4 m. This spacing is used to scale properties and parameters to account for the effect of the distribution of beams in the NS direction. The spacing was not used for modeling the NS-oriented strip footings for the interior frames. However, for the exterior frame it was introduced for the foundation elements as well, considering that the strip footing is oriented in the EW direction. The vertical loading included in the model was composed of 100% the gravity loading and 20% of live load, which was considered to be 3 kPa per floor. The input ground motions were obtained through a deconvolution process to calculate "within" motions; these motions were assigned to a rigid base in the FLAC model.

Table 1 shows the results of differential settlements obtained from the numerical analysis compared to the measured displacements after the Christchurch earthquake and after the 13 JUN 2011 event (measurements taken in March and July 2011, respectively, see Zupan 2014). The measured differential settlements represent the difference between the displacement at line 6 and the displacement at line 1 (see Figure 1) during the reconnaissance efforts (Zupan, 2014). The six different measured differential settlements in Table 1 (A to F) are for each line of columns in the EW direction (see Figure 1). Displacements were not measured after the Darfield event, because no significant damage was observed in the structure. It can be seen that the trend is capture in terms of displacements, being the results for the Christchurch event more severe than for the 13 JUNE 2011 event and the latter more severe than for the Darfield event, for which minor displacements were estimated.

Table 1. Comparison of differential settlements from numerical model and measured displacements after different events

Event	Calculated Differential Settlements (cm)		Measured Differential Settlements (cm)					
	Interior Frame (Lines B to E)	Exterior Frame (Lines A and F)	Line A	Line B	Line C	Line D	Line E	Line F
Darfield	0.6	0.7	No measurements taken for this event					
Christchurch	2.5	2.8	2.3	2.0	1.1	1.3	2.7	2.7
13-JUN-11	2.0	2.2	2.0	1.8	0.5	2.1	0.1	0.3

Conclusions

Soil liquefaction-induced building displacements cannot be estimated directly using simplified empirical procedures that estimate 1D post-liquefaction reconsolidation (volumetrically induced) settlements, because such procedures do not capture the important shear mechanisms involved in building settlements. A better alternative is to employ a SSI numerical analysis taking into consideration the variability in the ground motion and soil properties. The presented SSI analyses of the FTG-7 building are able to capture the observed trends in the seismic settlement measured in the three primary earthquakes of the Canterbury earthquake sequence. This was only accomplished after carefully calibrating the constitutive model such that the response of the preliminary 1D seismic site response analyses were in general agreement with the results from laboratory testing and the results from well-established empirically based simplified liquefaction evaluation procedures. After calibration of the soil model, reasonable SSI dynamic analyses could be performed that provided useful insights. The inability of continuum based soil models to capture the effects of soil ejecta should be recognized for cases wherein this particular mechanism governs performance. Additional research is warranted to investigate the seismic response of other buildings for the Canterbury earthquake sequence.

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