CASE HISTORY: USE OF VIBRO-REPLACEMENT STONE COLUMNS FOR BOTH LIQUEFACTION MITIGATION AND TO IMPROVE STATIC SETTLEMENT

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ABSTRACT

At a project site in San Diego, California, the hazards of liquefaction and seismically-induced settlement were present, in conjunction with relatively compressible soils that would require a low bearing value for use of spread footings. In addition, the site had variable conditions such that a portion of the site did not exhibit such hazards. Therefore, vibro-replacement stone column ground improvement was chosen to be performed such that the multiple hazards would be mitigated, allowing for use of spread footing foundations for a new building throughout the site. The post-improvement seismic performance of the soils beneath footings was calculated and served as the basis for seismic design for the vibro-stone columns. The post-improvement static performance of the soils beneath footings was evaluated using a modulus test in the field. The results of the modulus test confirmed the design assumptions regarding static settlement. The construction techniques are discussed herein, as are the testing and calculations used to justify the mitigated hazards and compliance with the design criteria.

INTRODUCTION

Turner Construction Company is building a new office building in San Diego, California, two stories in height, and with no subterranean levels. A plan of the site with the proposed building footprint is shown in Fig. 1. The site is underlain by older Mission Valley Formation in the southern portion and by young colluvial deposits in the northern portion. The approximate boundary between these two formations is shown in Fig. 1. The Mission Valley Formation materials consist of medium-dense and higher density granular materials and very stiff fine-grained soils. The younger alluvial/colluvial deposits at the north boundary of the site are poorly consolidated sandy, silty, or clay-bearing soils. Geotechnical explorations were conducted on the site, consisting of Cone Penetration Tests (CPTs), and borings that included Standard Penetration Tests (SPTs). The explorations located on the north side of the site encountered loose silty and clayey sand to depths of 50 feet below grade. Dense gravel/cobbles were interbedded in these upper deposits.

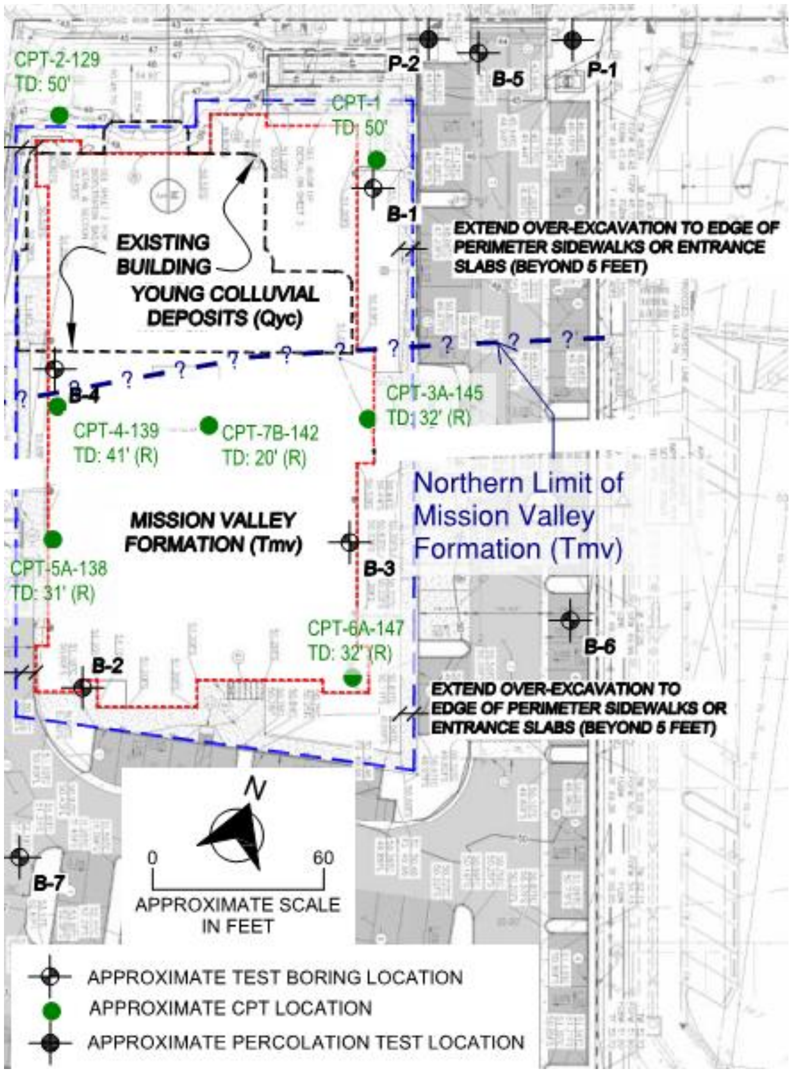

Fig. 1. Plot Plan

The soils as encountered in the explorations were interpreted based on the SPT blow counts as consisting of the following:

- In the northern portion of the site - alluvium/colluvium with a thickness of up to 35 feet, underlain by Mission Valley Formation
- In the southern portion of the site - Mission Valley Formation sediments starting at the ground surface.

Groundwater was encountered in two of the borings at the site at a depth of about 29 feet below grade, and pore pressure dissipation tests in CPT-1 indicated groundwater at a depth of about 25 feet below grade. The historic high (design) groundwater level was considered to be at a depth of 20 feet below grade, per the requirement of the Geotechnical Engineer of Record.

Some of the layers of alluvium/colluvium (in the northern portion of the site) were found to be liquefiable in the event of the Maximum Considered Earthquake (MCE) ground motion, with a peak ground acceleration 0.62g and associated Magnitude 6.89. The soil layers susceptible to liquefaction were found to include a layer from a depth of 38½ feet to 43½ feet below grade at the location of Boring B-1. The initial geotechnical analysis indicated a potential differential seismic foundation settlement of as much as 1 inch over a horizontal distance of 40 feet.

There is a slight slope to the ground surface at the site (a ground surface elevation change of about 5 feet over the length of the building). Based on the potential for liquefaction, and the ground surface slope, the site was considered to also have a low potential hazard of lateral spreading if the site grades were unchanged.

FOUNDATION SUPPORT DESIGN CONSIDERATIONS

Considering the potential for liquefaction and the presence of soft soils at the site which also would result in excessive static settlement of shallow foundations, traditional options for foundation support could consist of:

- Pile foundations, with piles designed to withstand down-drag loading due to seismic settlement, and designed to withstand the anticipated lateral spreading potential.
- Spread footings placed on improved ground using cementation, densification, or reinforcing techniques, such as
  - Deep Soil Mixing
  - Vibro-Replacement; and
- Intermediate Foundations consisting of spread footings placed on columnar elements such as rigid inclusions or semi-rigid inclusions. Intermediate foundations would need to be designed to accommodate both the static and seismic concerns with the site.

A request was made to a variety of firms for proposals for either ground improvement or intermediate foundations, with the potential to use any method that would meet the following objectives:

- Meet static settlement requirements of 1 inch with a differential static settlement of ½ inch between columns and ½ inch in 40 feet of wall foundation.
- Be designed to accommodate the anticipated seismic settlement (or mitigate the anticipated seismic settlement) to a differential of 1 inch in 40 feet (no total seismic settlement was required or considered).
- Provide a specified bearing capacity for spread footings (a design soil bearing pressure of 4,000 psf for dead plus live loads beneath interior columns, and design soil bearing pressure of 1,200 psf to 3,000 psf for the perimeter footings).
- Provide a solution acceptable to the geotechnical engineer of record, structural engineer of record, and the Authority Having Jurisdiction (AHJ).
- Be economical for the owner.

Based on the objectives presented as provided above, a variety of techniques were proposed. The chosen solution was built on the principle of optimization of improvement and focus on cost efficiency for the owner. Based on an evaluation of alternatives by different proposers, Malcolm Drilling Company (MDCI) was chosen to perform a stone column ground improvement program for the project. Since the site had much better characteristics at the southern portion than the northern portion, zones of different solutions could be provided. Solutions were applied such that the northern portion of the site had a higher degree of mitigation than the southern portion of the site.

SITE EVALUATION AND GROUND IMPROVEMENT DESIGN CONSIDERATIONS

A CPT-based liquefaction analysis according to Boulanger & Idriss (2014) was performed using GeoLogismiki CLiq 3.5.2.5 for all pre-improvement CPTs. The volume strain reduction factor was used based on the Cetin (2005) method with a conservative weighting depth limit at 80 ft. Both the liquefaction-induced settlement and dry sand seismic settlement were calculated, as shown in Fig. 2. Based on these analyses, only CPT-1 and CPT-2-129 showed appreciable potential for seismically induced settlement concerns.

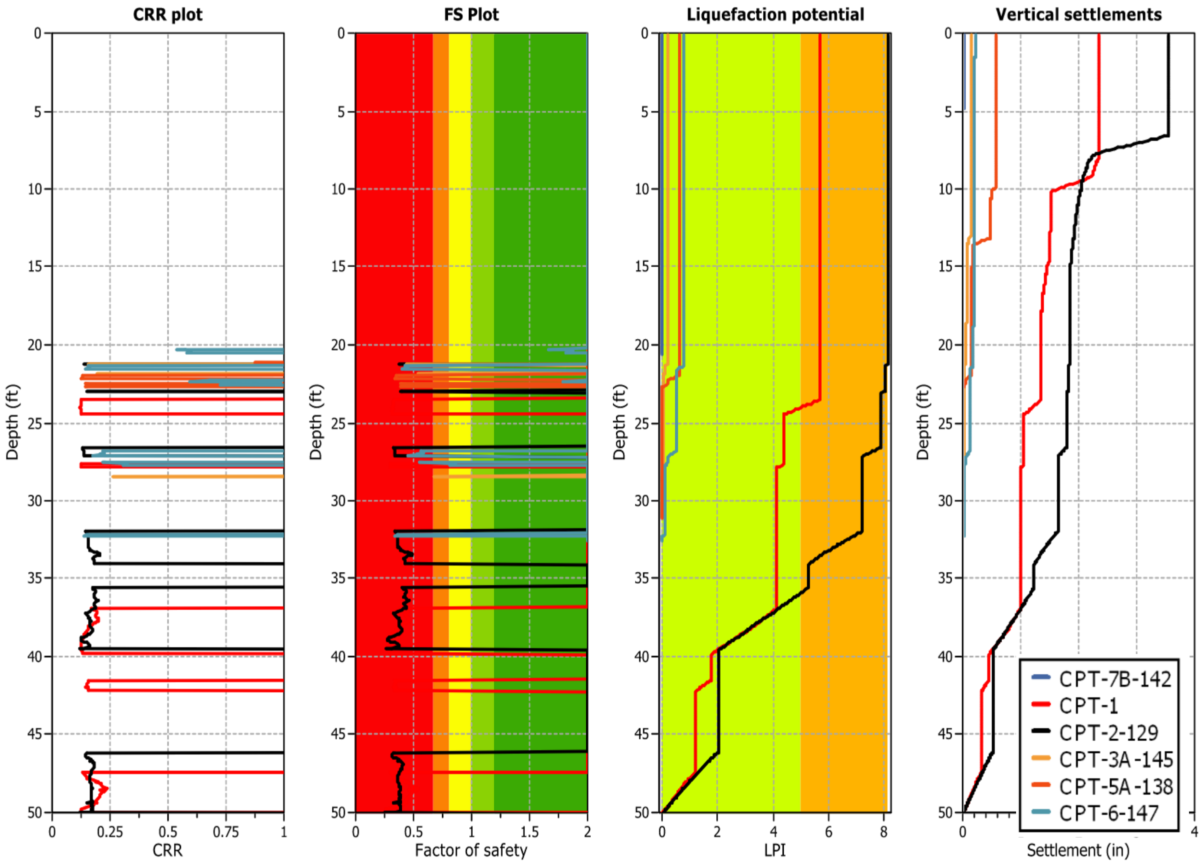

Fig. 2. Pre-Treatment CPT Seismic Evaluations

Baez (1995) describes a procedure to predict the densification by vibro-replacement stone column treatment as a function of pre-treatment CPT friction ratio and the stone column replacement ratio. Based on this procedure and MDCI's experience, the post-improvement CPT tip resistance profiles based on the pre-improvement CPT-1 and CPT-2-129, were predicted to be as shown in Fig. 3.

Following the same seismic analysis approach for the pre-improvement CPTs, the post-improvement seismic settlement was calculated based on the predicted improvements at the locations of CPT-1 and CPT-2-129, as plotted in Fig. 3 as well. All other pre-improvement CPTs, in the center and south portions of the building footprint indicated no or minimum liquefaction potential and did not require seismic improvement.

In addition, the constrained modulus, shear modulus, shear strength, shear strength ratio and over consolidation ratio (OCR) were predicted at the location of each CPT using the commercially available software CPeT-IT V3.7.1.2 by GeoLogismiki.

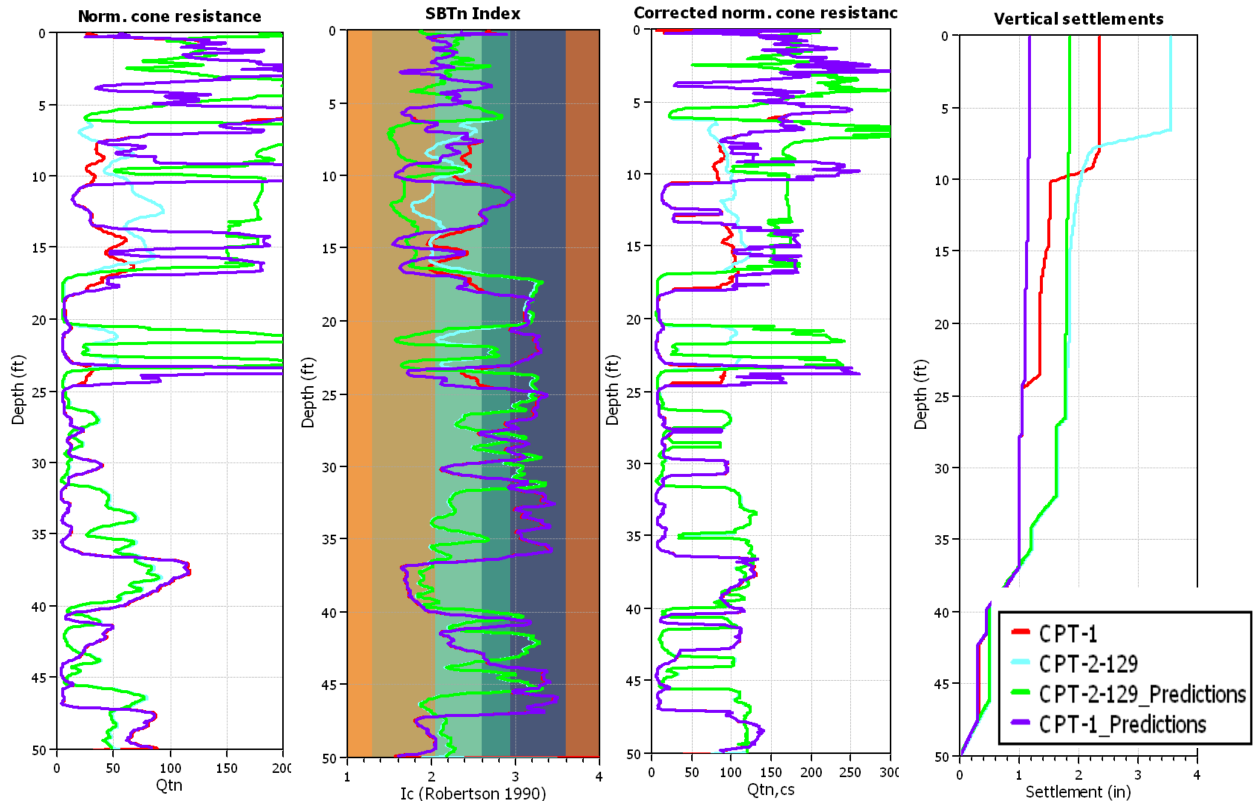

Fig. 3. Predicted Post-Improvement CPTs and Seismic Settlements

Based on the pre-treatment CPTs, CPT-3A-145 and CPT-4-139 have the greatest amount of clayey soils. Given the high OCR values in the footing stress influence zones, the induced stress in the soils due to the proposed footing load did not exceed the past maximum stress for the clayey soils. Therefore, the settlement calculation based on the soil's constrained modulus was determined to be applicable at this site and for this foundation design loading range.

A composite soil constrained modulus profile was developed based on the lower bound values of all pre-improvement CPTs. This conservative soil profile, ignoring any potential soil densification, was then used for the stone column static design for the settlement.

For the purposes of evaluating the settlement of the stone columns, MDCI evaluated two proposed stone column and soil modulus combinations, based on experience and typical assumed design values. The intent in selecting two sets of modulus parameters for the soil and stone columns for analysis was to evaluate the impact of changes in the design assumptions to the overall design treatment depths. From this, a robust and conservative treatment depth could be evaluated and selected.

The static settlement under the footings and grade beams for various treatment depths was calculated. The equivalent spring approach was used to calculate the stress in the stone column and soils. The footing stress attenuation as a function of depth below the footing was calculated based on a 2:1 (Depth : Horizontal Spread Distance) ratio, a conservative stress distribution approached commonly used in stone column design. The design footing static settlement was computed as the combination of settlement within the stone column treatment depth and settlement below the treatment depth.

After completing the seismic and static designs, a comparison of lengths was performed to determine which case governed the treatment depths. The seismic analysis showed that only moderate improvement (stone column treatment depth of approximately 11 feet) was required to meet the differential seismic criteria across the site. Thanks to the extensive investigation of the site, the seismic differential settlement criteria was demonstrated to be met based on calculating the anticipated residual seismic settlement between the boring/CPT locations. Ultimately, the static settlement criteria governed the stone column design lengths (stone column treatment depth of 18 to 30 feet). Accordingly, stone columns of varying lengths based on the loading, size, and location of the footings and grade beams were implemented in the final submittal, as shown in Fig. 4. These designs and plans were then reviewed with Turner Construction Company and Moore Twining & Associates (the geotechnical engineer of record).

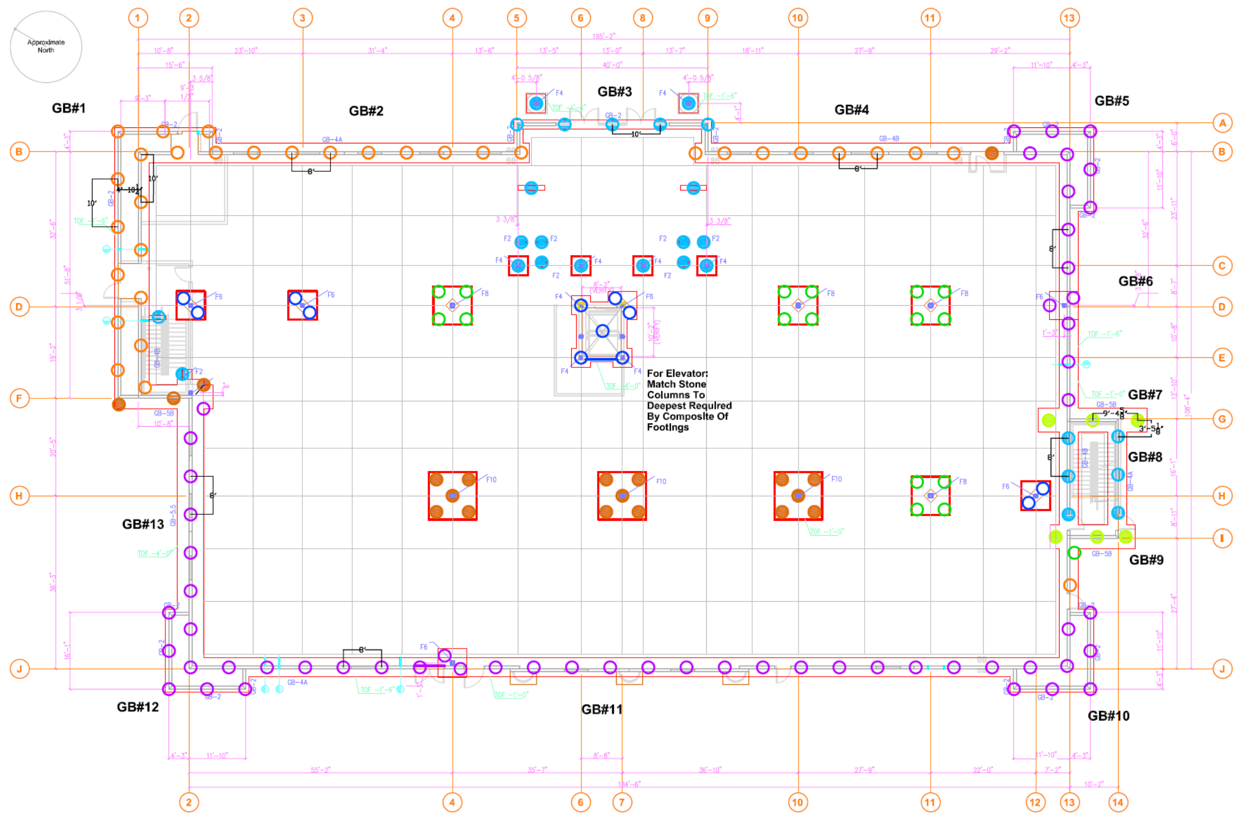

Fig. 4. Vibro Stone Column Layout under Building Footings

STONE COLUMN CONSTRUCTION AND VERIFICATION CONSIDERATIONS

The stone columns were installed with a bottom feed vibro system. The vibratory energy is generated by eccentric weights that rotate on an internal shaft near the tip of the vibrator. The vibrator and backfill pressure chamber are suspended from a crane as a single unit. The unit is lowered to the ground and penetrates by means of its own weight, vibrations, and air jetting. Once the design depth is reached, the vibrator is lifted in stages as the stone backfill is fed from a stone hopper through the pressure chamber and follower tubes, and expelled at the vibrator tip. For each stage, or “lift”, the vibrator penetrates the stone which expands the diameter of the column to approximately 2.5 ft, the column design diameter. These actions continue until the column is completed, as shown in Fig. 5.

The stone column backfill material was #57 crushed rock, meeting the following criteria: (1) 100% passing one inch sieve, (2) less than 5% passing #4, (3) durability index greater than 40.

Fig. 5. Stone Column Installation at Site

During construction, MDCI implemented a quality control program that included recording the depth, maximum amperage, and volume of crushed stone utilized. This allowed for the calculation of the theoretical installed diameter, and to confirm an adequate stone column replacement ratio was achieved.

For verification testing post-construction, a modulus (plate load) test was performed on one of the first production stone columns to confirm the design modulus and anticipated bearing capacity. The modulus test was performed based on methods modified from ASTM D1143 (Standard Test Methods for Deep Foundations Under Static Axial Compressive Load) and the former ASTM D1194 (Standard Test Method for Bearing Capacity of Soil for Static Load and Spread Footings, now withdrawn by ASTM due to a lack of reviewers for the past 10 years, however considered to still be valid). The procedure included applying loads in intervals up to 1.5 times the design load (1.5 DL) and holding each load for 1 minute. At 1.5DL, the load was held for 60 minutes while recording the displacements at planned time intervals. Then the stone column modulus from the applied load was back-calculated from the displacements at each load to verify and confirm the value used in the design of the stone column. The test exceeded the design modulus, which confirmed the design and quality of the stone columns as constructed.

Due to the tight project schedule and the achievement of a higher stone column replacement ratio than the design ratio, the footing differential seismic settlement criteria was taken to be satisfied based on the pre-CPTs and the calculation of post-improvement predicted results using the Baez (1995) method; no post improvement CPTs were performed. The following items provided an assurance of the design performance:

- The use of the worst-case soil profile was used in performing settlement analysis.
- The stone columns were installed to 1.6x or greater of the calculated length required to meet the differential seismic settlement requirements at the site.
- A high replacement ratio was utilized beneath the footings.
- The modulus test exceeded the design value.
- Structure elements not directly supported by the stone columns were not required to have seismic settlement mitigation (floor slab, surrounding improvements, etc.).

As such, once the stone columns were installed and the modulus test was completed, the site was turned over for final grading and installation of foundations.

CONCLUSIONS

Given the unique site's subsurface condition, the vibro-replacement stone column option was selected as the best improvement alternative to accommodate the project design loads and meet the settlement and life safety criteria along with the cost expectations of the owner. Vibro-replacement stone columns were designed to limit the total post-construction static settlement to 1 inch at column and wall locations; allowing for a differential static settlement of ½ inch between columns and ½ inch in 40 linear feet of wall foundations. The design was further refined by considering the design groundwater and Maximum Considered Earthquake (MCE) levels to limit the differential seismic settlement to 1 inch over 40 linear feet across the site. Finally, the design was refined to adjust the improvements based on the varying demands across the site.

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