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Compaction grouting solved a liquefaction potential problem in Oman

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ABSTRACT: Compaction grouting is a soil improvement technique that involves the injection into the ground of a very stiff mortar. The injected material does not permeate the native soil, but results in a controlled growth of the mortar bulb mass that displaces the surrounding soil. Soil improvement by compaction grouting is becoming more and more popular worldwide due to its flexibility, limited impact on the environment and cost effectiveness. In the field of earthquake geotechnical engineering, compaction grouting is now commonly applied to reduce the liquefaction potential of loose sandy layers. The paper describes the design steps adopted and the implementation of an intervention recently completed in Oman for reducing the liquefaction potential risk of a reclaimed sandy seashore before the construction of a new touristic complex.

1 INTRODUCTION

Compaction grouting technique involves the injection into the soil of a very stiff mortar that does not permeate the native soil, but results in controlled growth of the mortar bulb mass that displaces the surrounding soil.

The primary purpose of compaction grouting is to increase the density of soft, loose or disturbed soil formations, typically for settlement control, structural re-leveling, bearing capacity increase and mitigation of liquefaction potential.

As shown in Figure 1, compaction grouting firstly involves the installation into the soil of an 80-110 mm diameter casing to the required depth.

Subsequently, the stiff mortar is pumped through the casing at relatively high pressure (i.e. 40-80 bar) until one of the following limits is reached: (1) injection of a target volume, (2) achievement of a maximum pressure, (3) detection of undesired uplifts at the surface or distortions in neighboring structures.

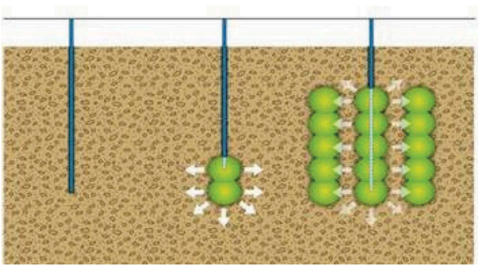

Figure 1. Scheme and principle of compaction grouting technique.

Grouting operations are usually carried out in 0.5-1.0 m length stages, in such a way to create a column of interconnected mortar bulbs. At each stage the soil particles are displaced radially from a growing bulb of mortar through cavity expansion effects into a close space, thus increasing the density of adjacent soil around the bulb.

Compaction grouting can be performed from the top to the bottom of the soil layer to be treated or, much more commonly, according to a "bottom-up" sequence, from the bottom of the treatment upwards.

According the final scope of the intervention, compaction grouting effectiveness is usually assessed by comparing the results of dynamic or static penetration tests or plate/zone load tests performed before and after the intervention.

1 LIQUEFACTION POTENTIAL PROBLEM AT "THE WAVE"

1.1 *Premises*

Liquefaction-induced foundation distortion and displacement during earthquakes continues to be a major cause of damage to all types of structures, including buildings, dikes, levees and seawalls.

As far back as the Niigata earthquake (1964). there is a clear historical evidence that ground improved sites suffer less ground deformation and subsidence than adjacent unimproved areas.

Case histories clearly indicate that ground improvement leads to a significant reduction, if not elimination, of large ground displacements during seismic cyclic loading.

The Kempinski Hotel and Apartments is a new touristic complex under construction at "The Wave", a recently developed residential area 30 km north of Muscat, the capital of the Sultanate of Oman.

The complex, composed by several buildings, insists on reclaimed sandy beach in front of the Indian Ocean. Among the others, the Apartments building, resting on an area of 60 m x 80 m and supported on conventional spread foundations, is the one closer to the beach shoreline (Figure 2).

Although in the area of Muscat the seismic risk is moderate, the potential liquefaction of the sands underlying the structure in case of an earthquake has been, since the beginning, a matter of concern for both Developer and Designer.

These worries led to a careful analysis about the potential liquefaction hazard of the involved soils and about the most cost-effective improvement method to be implemented.

1.2 *Soil investigation and potential liquefaction risk analysis*

The subsurface nature in the area of the Apartments building was investigated by performing Standard Penetration Tests (SPT) with core recovery and Cone Penetration Tests (CPT).

Figure 2. Aerial view and render of the new Kempinski Hotel and Apartments complex at "The Wave".

The resulting soil profile can be summarized as follows:

Table 1. Soil profile at Apartments building location.

From (m)	To (m)	Description
0.00	10.00	Very loose to loose, fine to medium SAND with abundant shell fragments
10.00	14.00	Medium to very dense, fine to medium, slightly gravelly SAND
14.00	30.00	Very dense, strongly cemented SAND

Groundwater table was detected 1 m below ground level.

The liquefaction potential of the upper sand layer was evaluated using the data of 2 CPT and the current calculation methods within geotechnical practice.

The peak ground acceleration (PGA) used in the analysis was 0.15 g, while a mean earthquake magnitude (M) of 6.0 was considered. The factor of safety (Fs) against liquefaction was calculated for each CPT location according to the procedure proposed by *Youd and Idriss* (1998), *Moss et al.* (2006) and *Haase et al.* (2011) and based on the "equivalent clean sand cone penetration resistance" ($(qc_{1N})_{CS}$), which accounts for fines content and overburden pressure.

The fines content was estimated from correlations to tip resistance and sleeve friction according to *Robertson and Wride* (1998) and was found to match well with laboratory sieve analysis performed on samples recovered from SPT sampler. Basically, the procedure for calculating the safety factor against liquefaction consists of following step by step calculation:

$$[CSR_M, q_C] \rightarrow q_{C1N} \rightarrow (q_{C1N})_{CS} \rightarrow CRR \rightarrow Fs \tag{1}$$

Where CSR_M = cyclic stress ratio developed in the cohesionless layers during the design seismic event, corrected via the Magnitude Scale Factor (MSF); q_C = measured tip resistance;q_{C1N} = normalized tip resistance, corrected according to the effective vertical pressure;$(q_{C1N})_{CS}$ = clean sand cone penetration resistance, which accounts for the fine content; *CRR*= cyclic resistance ratio; *Fs* = safety factor against liquefaction.

Resulting Fs for both CPT is shown in Figure 3(a). Lowest values of Fs were found between the depth of 2-4 m and 6-9 m below ground level, with an average value of 0.9 and a minimum value of 0.65.

The overall analysis led to the identification of a 10 m thick sand layer characterized by a moderate to high liquefaction potential risk (LPI = 2.36 – 6.11), according to the classification proposed by *Iwasaki* (1982) as modified by *Sonmez* (2003).

1.3 *Mitigation intervention design*

The calculation method has also allowed the definition of the minimum value of cone penetration resistance (qc_{min}) such as to reduce the risk to an acceptable value and increase the minimum safety factor to 1.25.

The method simply consists in evaluating CRR_{min}, qc_{min} and Dr_{min} values through back analysis, starting from the expected CSR_M and running backward the calculation steps of formula (1). This operation is carried out by assuming that the soil behavior index values (IC) does not vary by modifying the soil density, since the grain size of soil is unaffected.

$$[CSR_M,\ SF = 1,25] \rightarrow CRR_{min} \rightarrow (q_{C1N})_{CSmin} \rightarrow (q_{C1N})_{min} \rightarrow q_{Cmin} \tag{2}$$

Figure 3(b) shows the recorded qc and qc_{min} required to ensure a safety factor against liquefaction higher than 1.25.

Once the minimum tip resistance value (qc_{min}) necessary to avoid liquefaction phenomena is assessed, it is possible to calculate the corresponding minimum relative density Dr_{min} the

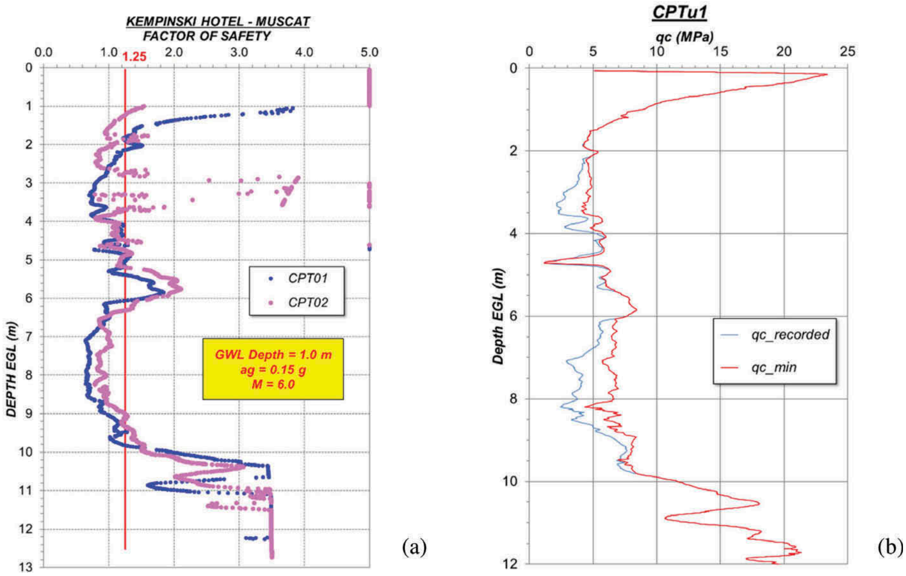

Figure 3. (a) Calculated Fs for CPT1 and CPT2. (b) Recorded qc and qc_{min} required to ensure a Fs > 1.25.

soil should acquire in order to satisfy the verifications. In this case, Dr_{min} has been calculated by employing the correlation proposed by *Jamiolkowski* (1988).

The increase in relative density of cohesionless soils is obtained through the injection of a stiff mortar via the compaction grouting method.

Compaction grouting columns diameter and their pattern/spacing to ensure the requested relative density, can be assessed by following the guidelines provided by *Mitchell* (1981), which are based on the following assumptions:

1. The injected mortar displaces the soil and, consequently, causes a decrease in voids volume exactly equal to its volume;
2. Displacement takes place in radial direction only.

If the average relative density of sand is increased from an initial value of voids ratio "e_0" up to a final value of voids ratio "e", it is possible to adopt the following ratio between the column spacing "S" and the column diameter "d":

$$S = \tfrac{d}{2}\sqrt{\frac{\pi\,(1+e_0)}{e_0 - e}} \tag{3}$$

The initial (e_0) and final (e) voids ratio can be assessed as follows:

$$e_0 = e_{max} - \frac{D_{ri}\%}{100} \times (e_{max} - e_{min}) \tag{4}$$

$$e = e_{max} - \frac{D_{rf}\%}{100} \times (e_{max} - e_{min}) \tag{5}$$

Where Dri% = initial relative density and Drf% = final relative density

In this analysis, maximum and minimum voids ratio e_{max} and e_{min} have been set at 0.965 and 0.34 respectively, according to typical values for cohesionless soils proposed by *Lambe and Whitman* (1979).

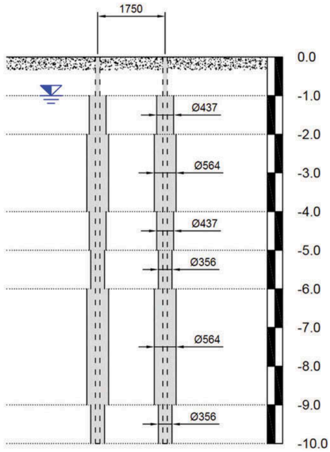

Figure 4. Design geometry of compaction grouting columns.

By setting the column spacing as 1.75 m, the minimum column diameter necessary to satisfy the verifications was calculated as 553 mm for CPT1.

However, such diameter was required only between 2 m to 4 m depth and 6 m to 9 m depth below ground level, within the loosest sand layer; therefore, a smaller column diameter was adopted for the remaining section of compaction grouting column.

The final design of compaction grouting column is shown in Figure 4.

1.4 *Design validation by preliminary trial field*

To validate the design assumptions, a trial field, consisting of 16 grout holes arranged in a squared grid pattern of side 1.75 m was conducted before starting production activities.

Mortar injection was sequenced to have primary (blue circle), secondary (red square) and tertiary (green triangle) grout holes, so that grouting was performed in a split-spacing pattern and adjacent locations were not grouted sequentially (Figure 5).

Two CPT (CPT-A) were executed before the column's installation and 4 CPT (CPT-B) after injection, with the scope of assessing the effects of compaction grouting. CPT-A1 was conducted within the test area, while CPT-A2 was executed 5m away.

Comparison between the average qc measured among CPT-B and both CPT-A1 and CPT-A2 is shown in Figure 6, which highlights general satisfactory results, except for 9-10 m depth and around 5 m below EGL, where the CG columns had a smaller diameter. Similarly, Figure 7 shows the safety factor against liquefaction before and after the treatment, and

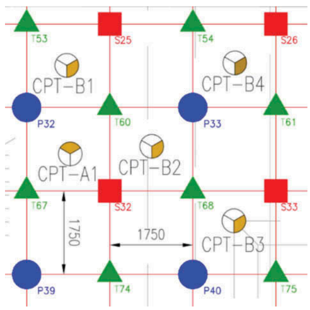

Figure 5. Plan view of trial field showing the CPT executed before (A) and after (B) injection.

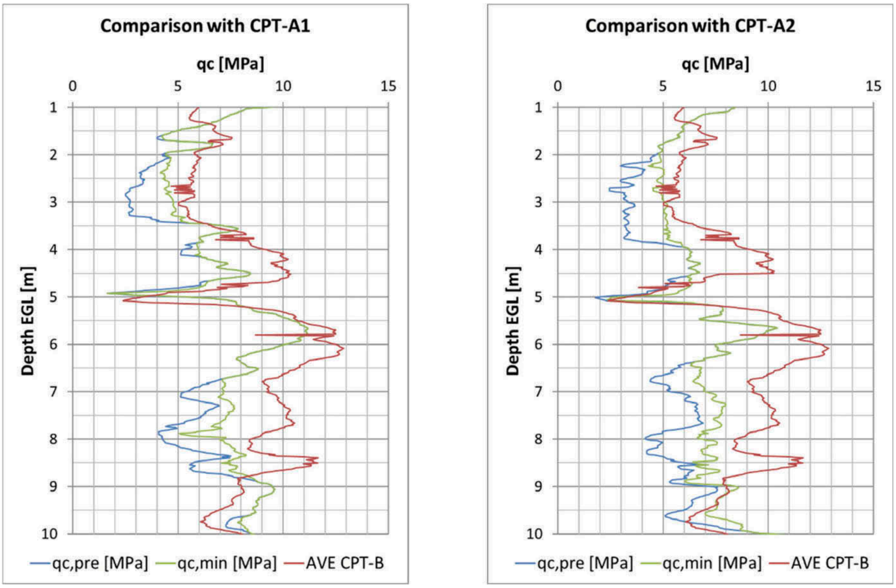

Figure 6. Comparison between CPT-A and CPT-B.

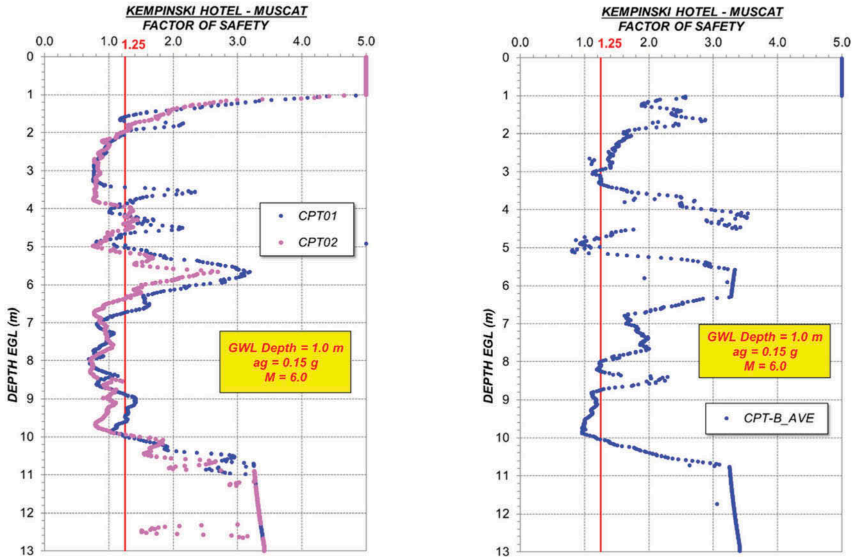

Figure 7. Fs before and after the treatment.

highlights that post-treatment Fs remains always around satisfactory values (> 1.25) except in correspondence of very limited areas, at the same depths where the column had a smaller diameter.

Most probably, this is related to the fact that the key assumption n. (2) in previous section 1.3 is not exactly true when there is a sharp change in column's diameter. In these cases, some displacements take place even in vertical direction due to possible border effects.

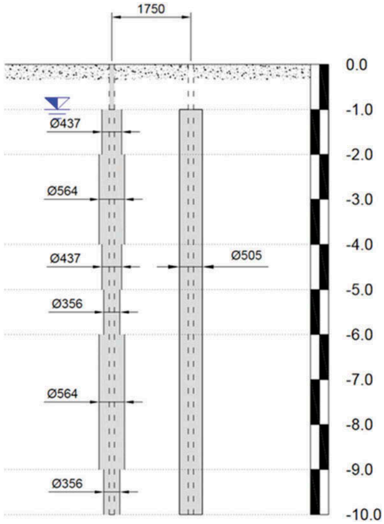

Figure 8. Original and new design geometry of compaction grouting columns.

Based on the trial test results, a new compaction grouting column geometry was proposed, with the scope of increasing the Fs to a minimum value of 1.25 along the whole depth of treatment.

2 INTERVENTION IMPLEMENTATION

2.1 *Production activities*

Production activities at site, which lasted 41 working days on a 10 hours/day basis, were carried out using two independent groups of equipment, each group consisting of one drilling rig type SOILMEC SM-8 for the installation of the casings, one drilling rig type SOILMEC SM-14 for the injection of the mortar and the retrieval of the casing and one concrete pump type PUTZMEISTER P-715.

The ready-mix mortar, constituted by a homogenous mix of cement, sand, water and conventional concrete additives, was supplied via the site by truck-mixers. Routine quality

Figure 9. General view of the jobsite and compaction grouting equipment in action.

Figure 10. Routine quality controls on the ready-mix mortar delivered at site.

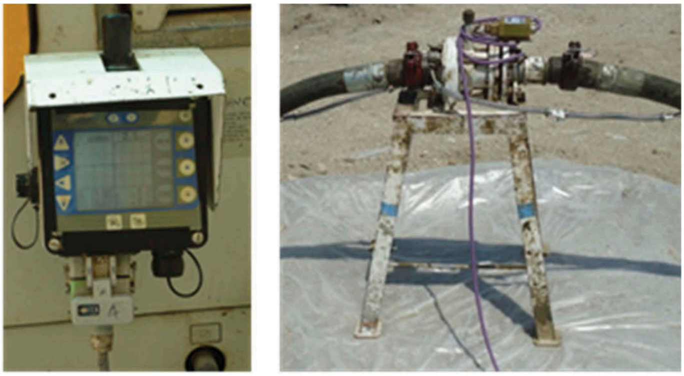

Figure 11. Automated data acquisition and processing system and pressure sensor.

controls on the mortar included the measurement of the temperature and the evaluation of the consistency (slump test by Abrams cone) for each truck-mixer arriving at site.

While grouting, volume, pressure and flow rate at each stage were monitored and recorded in real-time by an automated data acquisition/processing system type Jean Lutz BAP160, mounted on the concrete pumps. For pressure measurements, the system made use of a high sensitivity pressure sensor located at a maximum distance of 5 m from the hole under grouting.

The use of such a system led to a systematic step-by-step analysis of the response of the ground under injection that allowed for efficient decision-making by technical supervisors.

The key figures of the intervention can be summarized as follows:

Treated surface:2 106 m^2
Treated soil volume: 18 958 m^3
Total no. of grout holes: 688 no.
Total injected volume of mortar: 906 m^3
Average volume per grout hole: 1.32 m^3/hole
Average volume per meter: 0.15 m^3/m

2.2 *Final results*

The effectiveness of the intervention was finally evaluated by performing 10 nos. post-treatment CPT (CPT-C), randomly distributed within the site. On the left side of Figure 12 it is represented the net improvement between CPT-C and CPT-A, with reference to the minimum qc required as

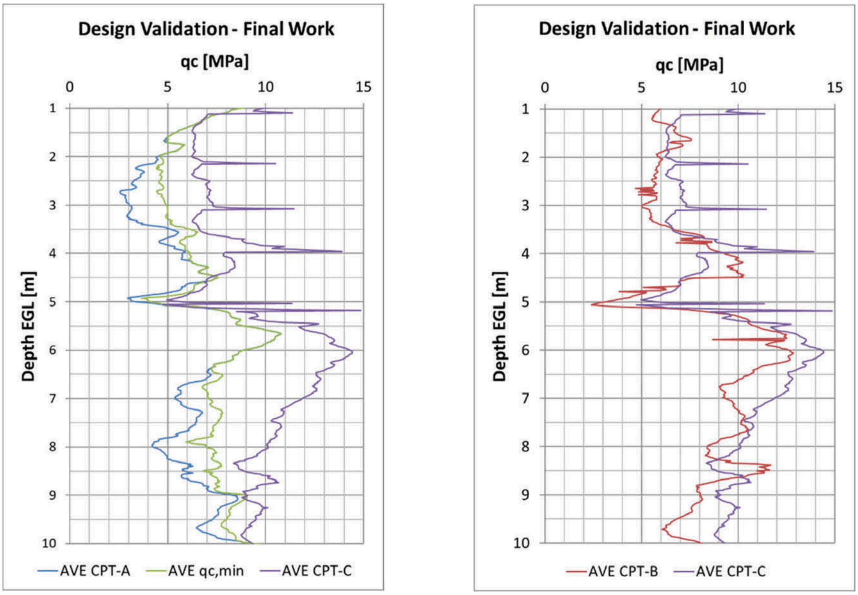

Figure 12. Comparison between CPT-A CPT-B & CPT-C.

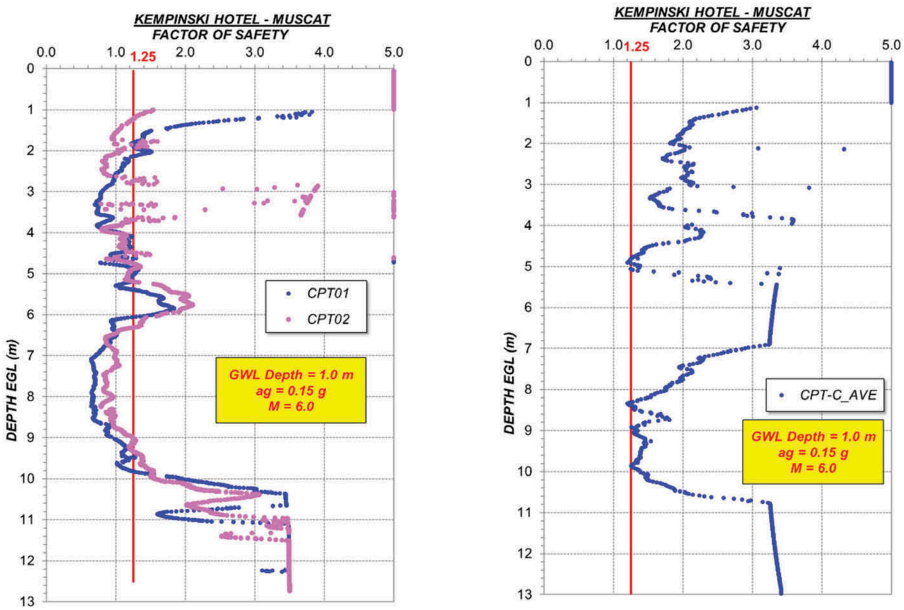

Figure 13. Comparison between pre and post-treatment safety factor (CPT-A and CPT-C)

per design. The graph on the right side of Figure 12 represents the comparison between the CPT-B (trial area – old column) and CPT-C (work completed – new column) and it shows that the new column ensures a more uniform improvement and substantially better improvement.

Finally, the Safety Factor against liquefaction pre and post treatment is represented in Figure 13. Fs calculated based on CPT-C (post-treatment) remains always around satisfactory values (>1.25) except in correspondence of very limited areas (10cm thick) at the depths of 5m and 8m below EGL, where it reaches the minimum value of 1.20. Results were considered satisfactory.

3 CONCLUSIONS

The intervention at "The Wave" has confirmed once more the flexibility of this soil improvement technique for reducing the liquefaction potential risk of reclaimed sandy formations in a cost-effective way and a very limited impact on the environment.

As matter of fact, the Client has evaluated in around 20% the cost savings in using this technique compared to possible alternative solutions (i.e. stone columns or vibroflotation).

Certainly, a Proper Design Approach, a Comprehensive Full-Scale Preliminary Trial Field and a Strict Quality Control are Necessary and Fundamental Factors for the Success of the Intervention.

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