Condition Assessment of Existing Reinforced Concrete Foundation – Case study of HRSG Foundations at DUBAL

Jaykumar Shukla[1], Shirshendu Chakraborty[2], Sambuddha Dasgupta[3]

1) Senior Geotechnical Engineer, CSAD L&T-S&L, Vadodara
2) Site Manager, F9BE HRSG replacement project, L&T-S&L, Vadodara.
3) Project Director, F9BE HRSG replacement project, L&T-S&L, Vadodara.

Abstract

With intention to overcome the limitations of the existing equipment in the downstream of upgraded GE frame 9B gas turbines, five HRSG structures are to be replaced at Dubai Aluminum Ltd., Dubai. Considering the size limitation of the site area and existing Turbine locations, complete removal of old HRSG structure and its existing concrete foundations may require more construction time. It was therefore decided to carry out condition assessment for existing reinforced concrete foundation which was built in 1980 for its service life extension. Present paper describes the condition assessment program and its outcome for the existing HRSG foundations, which can be referred for similar condition assessment works. After condition assessment, its serviceability was checked for next 30 years of operation. With suggested modifications, the foundation was then checked for new sets of loading conditions arising from installation of new HRSG structures. It was observed that the existing Reinforced Cement Concrete (RCC) foundation can safely support the new HRSG installations for its next service life. Presently new HRSG structures are under commissioning which are founded on the existing foundation structures.

1.0 Introduction

With structures are aging, the assessment of buildings, bridges, tunnels, dams and industrial structures is becoming increasingly important. Structural codes have been developed for new design, but they often are not appropriate for assessment of existing structures since there are significant differences between design and assessment. Structural reliability assessment is imperative in the cases where there is an extension of the service life and load/ actions are considerably changed. Usually for concrete structure of industrial plants, the life of the structure is assumed to be 25 to 30 years. However, it would very challenging to propose new structure founding on the structure built in year 1980 (almost 34 years back). There can be many doubts regarding the structural integrity of such structures. Since, any evaluation will involve engineering judgment and contains factors that cannot be readily defined and standardized. Moreover, being an international project, the joint agreement among all decision making engineers are also important.

2.0 Site information and existing conditions

The Project site is at DUBAL (Dubai Aluminum Limited) Aluminum Smelter Complex at Jebel Ali in UAE. The captive power plant at DUBAL complex, Dubai consists of combined cycle units and cogeneration units to generate electrical power for smelter and other facilities and process steam for desalination plant. DUBAL is carrying out GT Upgrade in five (5) of their GE frame 9B gas turbines in order to improve the power output and thereby overall power reserve of the plant. The GT upgrade is expected to improve the power output of the gas turbines from 80 MW to 88 MW at 35°C ambient. The GT upgrade, called frame 9BE, will allow higher firing temperature up to 2055°F, improving power output and heat rate. The GT upgrade of all five units is expected to increase the reserve capacity of the plant by 40 MW from the original capacity. The upgraded gas turbines (GT 9, GT 11, GT 12 & GT 13) are currently being operated at a lower firing temperature of 1965°F. The firing temperature could not be raised to the maximum firing temperature of 2055°F (1125°C) due to the limitation in the GT generator, associated electrical equipment and boiler (HRSG). The GT upgrade is expected to give a life extension of 25 years to the frame 9B gas turbines. Limitations of the associated equipment and to suggest the mitigation measures were studied to overcome the design limitations of the equipment, so as to operate the GTs at base load and maximum firing temperature of 2055°F. Based on the study, it was recommended to replace five HRSGs and associated de-aerator and feed water system along with necessary piping and instruments capable of meeting the requirements of new HRSGs. While for feed water structures, separate land area are available, for installation of new HRSGs in the proximity of the existing upgraded gas turbines, it was recommended to use the existing foundation footprint only.

3.0 Investigation program

As part of comprehensive condition assessment program, combinations of field tests followed by laboratory investigations were planned. Field tests like Ultrasonic pulse velocity (UPV) investigation of existing concrete, extraction of concrete cores from foundations, rebound hammer tests, pachometer test for reinforcement locations etc. were carried out. Laboratory tests like petrographic analysis, ionic chromatography, x-ray diffractomery, CO2 penetration and compressive strength of concrete cores are performed on the collected core samples.

4.0 UPV tests

Each HRSG foundation is divided in to the grid of approximate dimension of 1.5 X 1.5 m in order to locate each test at appropriate locations. The proposed grid marking are illustrated in the Fig. 1. Fig. 2 illustrates the UPV investigation going on the existing concrete foundation for one of the HRSG structure. Since the foundation footprint (14 x 40 m) is large, the response of UPV tests are imposed over the foundation layout grid (Fig. 1)

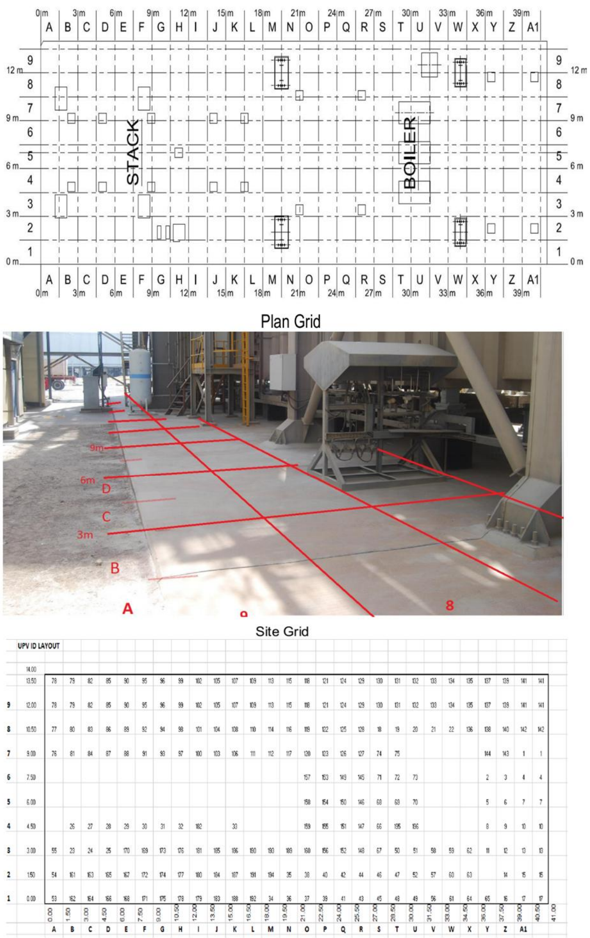


Fig. 1 Existing Foundation layout and illustrated grid marking.

For each foundation, approximately 190 UPV test results are obtained. Each observation is taken as the average of the three observation taken by varying the spacing between the UPV probes (transmitter and receiver). The responses of observed UPV for one of the foundation are presented in the Fig. 3. Observed wave velocity was in the range of 1000 m/sec to 5614 m/sec for one of the foundation which indicate that foundation has variable strength across the foundation footprint. Due to existing equipment accessibility in certain areas of the foundation was limited which could not be tested as highlighted in the Fig. 3.

Fig. 2 Measurement of UPV tests on each specified grid.

5.0 Extraction of Concrete Cores

Since the extraction of concrete cores are considered to be destructive type of the tests, it was jointly decided to obtain limited number of the concrete. Since main intention of obtaining concrete cores is to investigate concrete compressive strength, 100 mm dia concrete cores are drilled from the foundation footprint. On each foundation, four (4 nos) representative concrete core samples were collected which were then send to laboratory for further investigations. Typical concrete core obtained from one of the foundation is presented in the Fig. 4.

Fig. 4 Extracted core from HRSG No. 11

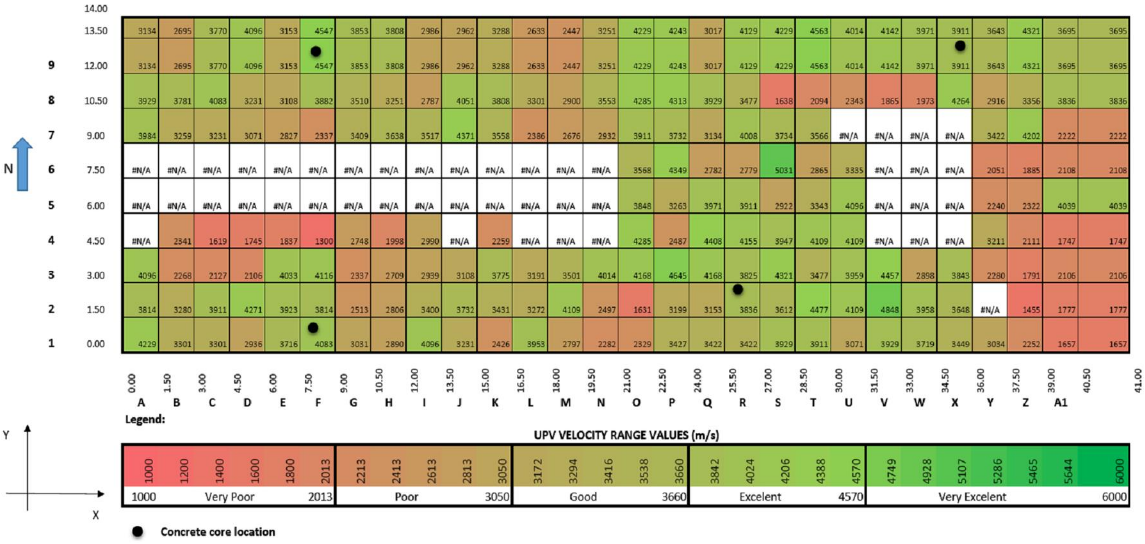


Fig 3 Typical UPV test response (wave velocity in m/sec) imposed over the foundation footprint; concrete core locations are shown indicatively

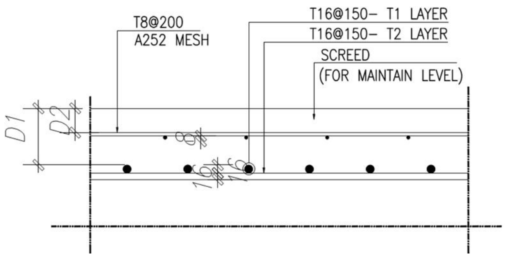
Though it was preferred to drill the core in the location where it can avoid the main reinforcement of the existing foundation, some places, drilled core encounter the reinforcement as well as illustrated in the Fig. 5. It accidently provides many more information regarding the reinforcement existing within the foundation. The structural drawing available for these foundations (see Annexure A) suggested that the 16mm dia reinforcement bars were used as main reinforcements. The reinforcement pieces obtained from the concrete cores calibrated the reinforcement diameter placed within the foundation there by giving indirect indication of the correctness of the available structural drawing.

Fig. 5. Concrete core encountering main reinforcements. .

While extracting the concrete cores, it was observed that from the top surface approximately 60mm below, 8mm reinforcement was encountered as shown in the Fig. 6. The condition of the reinforcement was also not so good and it was felt that the top reinforcement may not be in adequate condition. However, such reinforcement were not described in the available structural drawing. Hence it was decided to expose top surface till main reinforcement is visible. Typical exposed surface is presented in the Fig. 7.

Fig. 6 Top 8mm reinforcement observed in the concrete core.

It was observed that a screen reinforcement mesh 8mm dia 200mm c/c bothways were provided as protection layer having top concrete cover as 60 mm. Main reinforcement mesh of 16mm dia bars at 100mm c/c bothways were provided under the screen reinforcement mesh having effective cover of 100mm or more. Such arrangement was available for all foundations which cleared the doubt regarding available main reinforcement. Some of the locations surface cracks were observed and extents of such cracks were visible in the concrete cores as well. It was observed that the extents of such cracks were limited to the top screen surface only and no crack was observed to be penetrated up to main reinforcements in any of the existing foundations. Moreover, the widths of the cracks were not significant which lead splitting of the concrete cores refer Fig. 8. However, areas were cracks were visibly significant, it was decided to chip off the surface concrete and grout the areas.



MESH WIRE REINF.

SI Nr.	Diam. (mm)	Cover " D2", (mm)
1	7.67	64
2	7.77	69
3	7.76	72
4	7.81	75
5	7.97	72
6	8.27	77
7	7.99	80
8	7.88	83

MAIN REINFORCEMENT

SI Nr.	Diam. (mm)	Cover " D1", (mm)
1	17.94	117
2	17.59	104
3	17.60	116
4	18.01	110
5	17.81	128
6	18.16	132
7	17.54	120
8	15.62	133

Fig. 7 Main reinforcement and top screen reinforcement observed at the top of the foundation with measured cover.

Fig. 8. Typical cracks observed in the top portion of the concrete core.

Obtained concrete cores also provided the opportunities to get some other insights (see Fig. 9) as well. Overall, extraction of these cores provided sufficient information to characterize the in situ concrete condition for many parameters related to condition assessment.

Fig. 9 Surprizes in concrete cores

6.0 SONREB procedure

SONREB comes from the words SONic REBound. Both ultrasonic pulse velocity and rebound hammer measurements can be correlated to compressive strength. (EN 13791). The SONREB method, is a method of combining an ultrasonic pulse velocity measurement with a rebound hammer measurement to give increased accuracy to compressive strength estimation. Several studies were done by various researcher to correlate ultrasonic and rebound hammer. Below indicated are correlation curves recommended by various authors based on experimental specimen and mathematical calculation

Correlazione	*Autore*
$R = 7.876 \cdot 10^{-19} V^{4.636} I^{1.747}$	Lenzi, Versari, Zambrini (2010)
$R = 7.695 \cdot 10^{-11} V^{2.60} I^{1.40}$	RILEM – NDT4 (1993)
$R = 1.2 \cdot 10^{-9} V^{2.446} I^{1.058}$	Di Leo e Pascale (1994)
$R = 1.51 \cdot 10^{-7} V^{0.8084} I^{1.8815}$	Masi (2005)
$R = 8.06 \cdot 10^{-8} V^{1.85} I^{1.246}$	Gasparik (1992)
$R = 0.9 \cdot I + 0.022 \cdot V - 94$	Tanigawa, Baba , Mori

The above constant can be utilized if there was limited data gathered on site similar to the project site. Hence for this project, we were able to carry out compressive strength of core sample, Rebound hammer and UPV test correlation between which was further used using the below equation. Compressive strength $F_{ck} = aV^bS^c$

Using compressive strength observed from laboratory tests, UPV tests results, is possible to calculate the property of individual constant describe by equation below a = 48814.5

b= -0.2141

c= -1.355

V= ultrasonic pulse velocity in m/sec

S=Rebound value (reduced value)

7.0 Other tests

Chromatographic analysis doesn't show exceeding chlorides, Nitrates and Sulphate. Diffractometric analysis detects only a slight amount of Ettringite in few samples, not potentially harmful to the concrete. Petrographic analysis from thin slide didn't observe alkali-carbonate reaction or alkali-silica reaction. Few samples showed calcite incrustation on the gravel surface. The description of the entire procedure is beyond the scope of present paper, however, mircorscopic illustration is presented in the Fig. 10. The carbonate penetration was further extrapolated for further 30 years of service life (future service years) however found lover than the actual concrete cover provided for the main reinforcements i.e. 100mm.

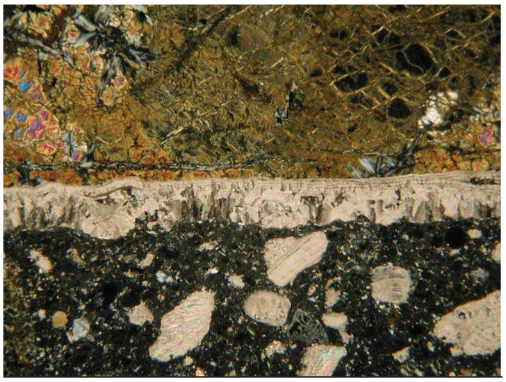


Fig. 10. Calcite incrustation on gravel surface. Microphotography at polarizing microscope, transmitted light thin section

8.0 Conclusion & Recommendations

For UPV Tests, it was found that average reading was 3298m/s or 3.298km/s but few readings were lower than expected due to cracks on surface which leads on lower velocity. Considering the tests were done using indirect method, equipment reading can be expected to be lower by 5-20%. However, the concrete core sample amplified an average velocity reading of 4630m/s or 4.630km/s using direct method. Meanwhile, few perpendicular cracks were observed 22mm deep from top of concrete but not serious hence is didn't reach main reinforcement of foundation slab.

- Chromatogrpahic analysis shown less quality and do not exceed permissible level of chlorides, nitrates and sulphates. Lower level of chlorides oxidation was not harmful to current concrete structure.
- Diffractometric analysis found less quantity of Ettringite but not potentially harmful on concrete structure while presence of Portlandite indicates that the concrete specimen were not carbonated (except the top layer average -23mm).
- Carbonate analysis was found at range 11-33 mm from top of concrete specimen and not reached to reinforcement level. With high level of carbonation depth subsequent high Rebound read will be affected. Also low UPV velocity can be derived due to superficial cracks at top level of concrete and crack mapping can be observed. A reduction factor of 0.7 shall be applied on original rebound value in order to arrive with reduced value of rebound hammer.

Compressive strength of concrete were high enough with range between 46 to 50 N/m^2. SONREB procedure was employed and final concrete strength for design was factored by 1.5 in order to use conservative estimate of the average concrete. For SONREB and future testing on HRSG foundation (typically No. 11) subsequent data were established to correlate UPV and Rebound hammer Following correlation found appropriate and representative of the concrete strength

Compressive strength $F_{ck} = aV^{b}S^{c}$

Where a = 48814.5
b= -0.2141
c= -1.355
V= ultrasonic pulse velocity in m/sec
S=Rebound value (reduced value)

As per laboratory validation the result of high carbonation and low reading of UPV on grade slab is only superficial due cracks and deterioration coming from the age of concrete structure. However, in general, the existing structure are still in good condition requiring localized repair works. Final commission is still going on and Fig. 11 illustrates the arrangement of the anchor bolts for the repositioned support structures for the new HRSG structures.

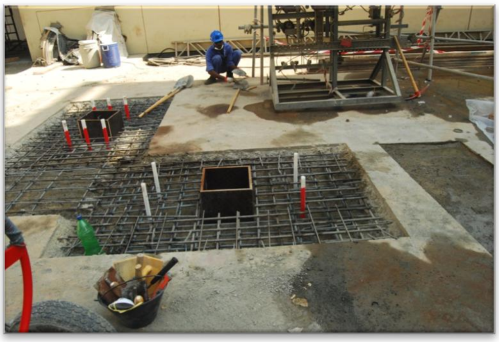
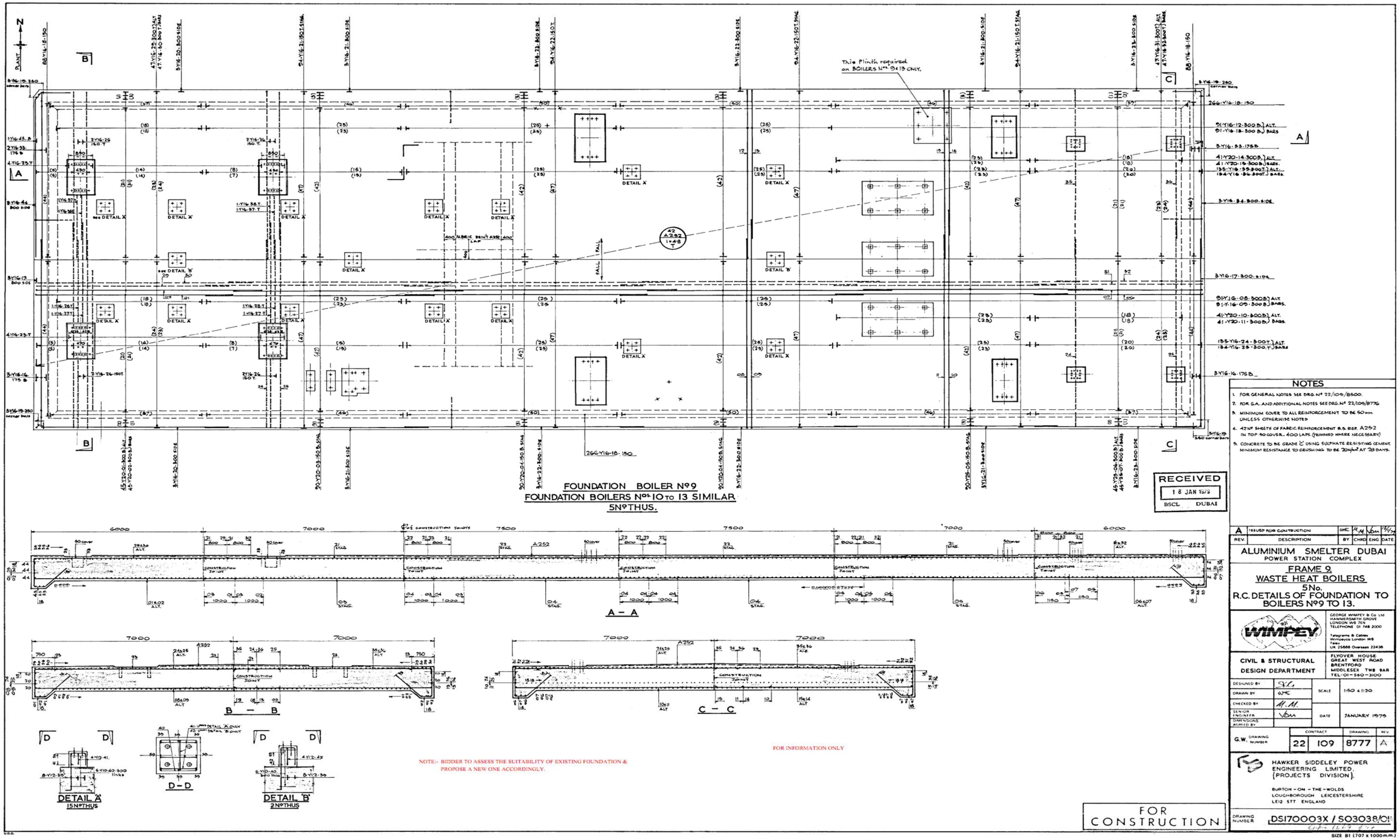


Fig. 11 Modification and installation of the new post install anchor bolts for new HRSG support structures.

9.0 References

RILEM NDT 4 (1993) Recommendation for in situ concrete strength determination by combined non-destructive methods.

ASCE SEI 11 (1999) Guidelines for structural condition assessment of existing structures. ASCE Standard.



Annexure A – Structural Drawing of existing HRSG foundation (illustration of reinforcement layout)