A New Building Inflicted by the Kobe Earthquake of January 17, 1995 and the Measures Taken to Restore and Strengthen the Foundation-Building System

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A NEW BUILDING INFLICTED BY THE KOBE EARTHQUAKE OF JANUARY 17, 1995 AND THE MEASURES TAKEN TO RESTORE AND STRENGTHEN THE FOUNDATION-BUILDING SYSTEM

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A new building in the city of Nishinomiya, near Kobe, was just ready for hand over when the January 17, 1995 earthquake occurred. The building looked structurally intact, but the piles supporting it were found to have been completely damaged. A comprehensive plan involving a tricky simultaneous operation of 61 hydraulic jacks was developed and successfully implemented to jack-up and restore the foundation-building system. Luckily, there were no aftershocks closer in size to the main event during the implementation phase. The differential settlement and tilting were practically nil when the restoration was complete. The experience gained from the project is vital in future restoration and reconstruction of pile foundations damaged by extreme earthquake events.

1 Introduction

The magnitude 7.2 earthquake that rocked Kobe and the surrounding Kinki region of Japan on the morning of January 17, 1995 at 5:46 a. m. caused widespread devastation that left thousands dead and caused property damages reported (1995) to be 140 billion US dollars. Japan Meteorological Society (JMA) assigned for the first time the highest intensity of 7 to a region about one kilometer wide. Figure 1 shows the region of about 22 km in length extending from Suma ward of Kobe city to the Nishinomiya city, along the Suwayama and Gosukebashi fault lines, where the damage was reported (1995) to be strongly concentrated.

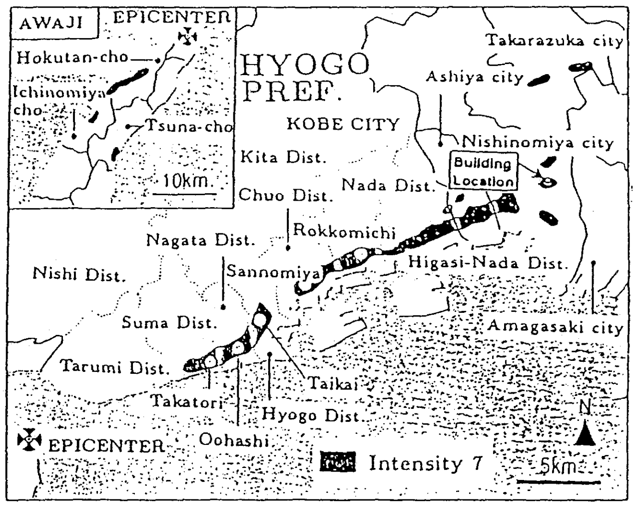

Figure 1: Area with JMA Intensity 7 and the Location of the Building

Buildings at different stages of construction were affected by the earthquake. A seven storied reinforced concrete condominium building in the city of Nishinomiya, lying in the region of intensity 7 shaking as shown in Figure 1, was just completed when the area was pillaged by the earthquake. Severe damage to the precast concrete piles supporting the building, which was not immediately evident initially, was confirmed from subsequent investigation. A comprehensive plan to restore the building was developed and implemented successfully.

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The general flow of the restoration project is shown in Figure 2. In this paper, the details of the various phases of the restoration process are discussed and the difficulties encountered during implementation phase are highlighted. The methodology utilized and the experience gained from this rather unique restoration project is expected to be a valuable addition to the background information towards formulating requirements for the design and construction of pile foundations for extreme earthquake events.

2 The Pile Foundation

Most of the column footings of the condominium building consisted of three prestressed precast high strength concrete (PHC) piles under them as shown in Figure 3. Prestressed high strength concrete piles reinforced with ribbed steel bars, generally referred to as PRC piles in Japan, were utilized at the four corner footings of the building.

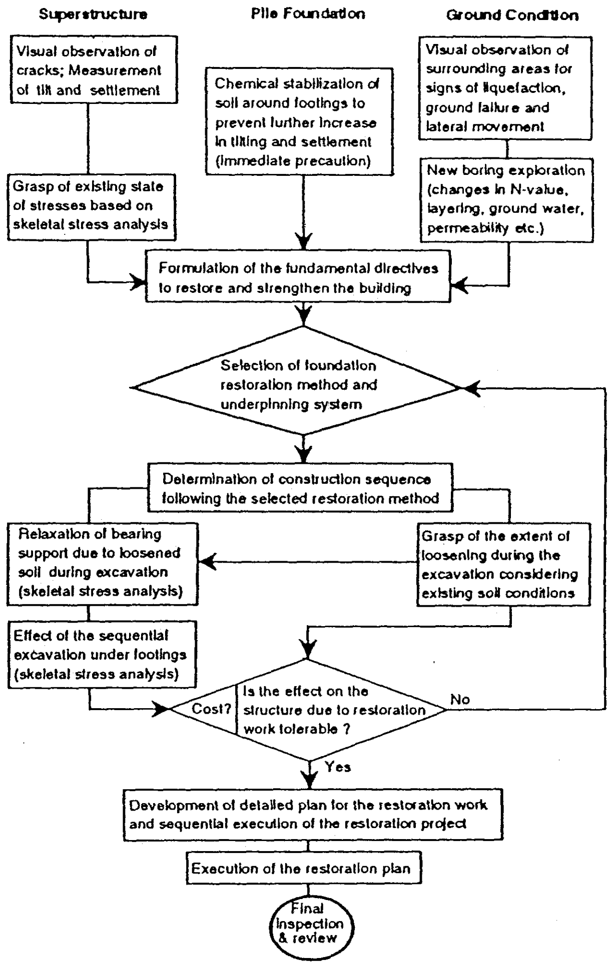

Figure 2: Flow of the Restoration Plan

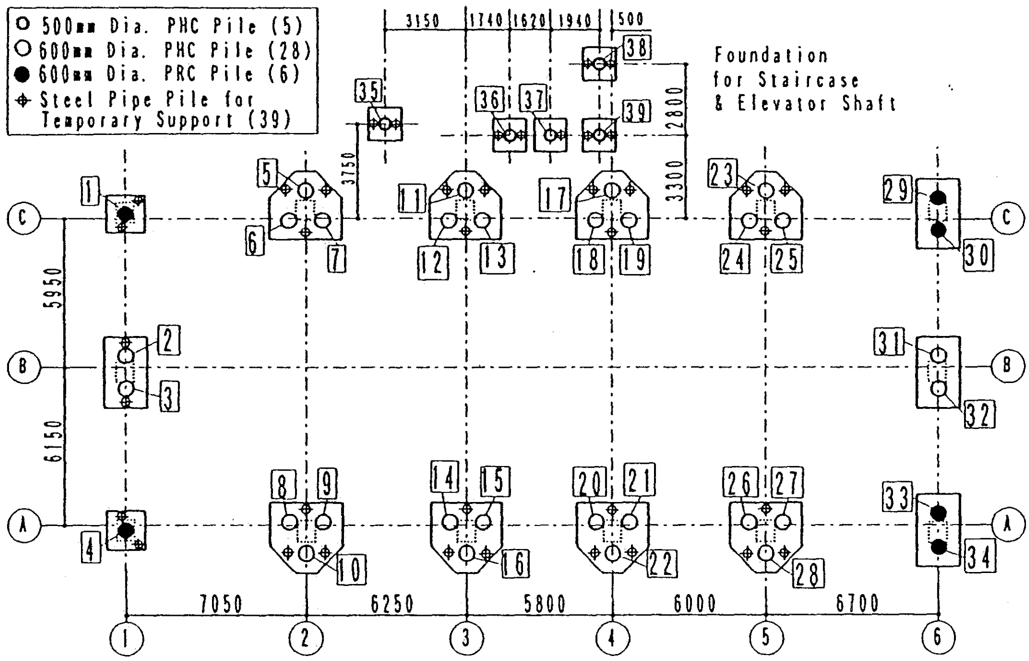

Figure 3: Building Foundation Plan Showing the Location of PHC and PRC Piles and Footings. The Crossed Circles Show Location of Steel Pipe Piles for underpinning.

The integers 1 to 39 inclosed by rectangles in the foundation layout of Figure 3 are the sequential pile numbers. All the piles supporting the building are of 600mm external diameter, except for the six supporting the staircases and the elevator shaft (numbered 35 to 39), which are of 500mm. The respective locations of PRC and PHC piles are also indicated in Figure 3. The Cross-sections of the ϕ600mm PHC and PRC piles, together with that of ϕ500mm PHC pile are shown in Figure 4. The prestressing strands, helical shear reinforcement and longitudinal rebars where applicable, are also given. All the piles were 9.0m long, except for the four numbered 36 to 39 in Figure 3, which were 8.0m long.

The piles were installed by the so called pre-boring method. Many variations of this method are practiced in Japan, where stringent regulations against noise pollution and vibration transmission make pile driving practically unthinkable around built-up areas. The method utilized in this case consisted of using an auger-head and shaft combination to drill a hole together with the injection of cement slurry, such that the cement slurry gets mixed with the soil as the drilling proceeds.

The outer diameter of the auger head was about 100mm larger than the pile diameter. When the drilling reached the required depth, the auger was withdrawn while continuing the mixing operation. Then the open ended precast pile was inserted into the hole. The soil-cement paste fills the inside and the outside of the pile, and a portion of the paste is displaced as the pile is inserted. The pile gets bonded to the surrounding soil as the soil-cement paste hardens and gains strength.

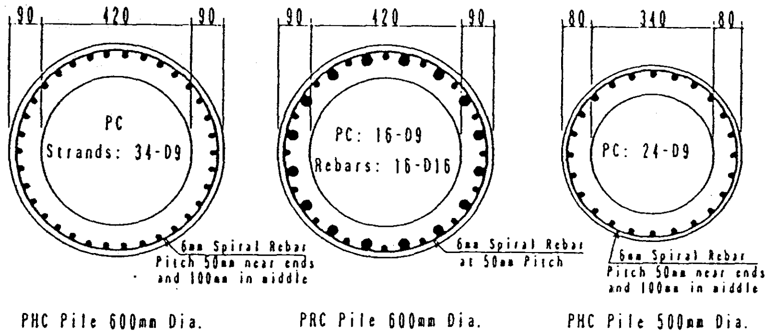

Figure 4: Cross-section of PHC and PRC Piles. (Prestressing strands, spiral ties and rebars are also shown.

3 Inspection After the Earthquake

From the initial visual inspection immediately after the earthquake, the structure was thought to be practically unscathed as seen in Figure 5. However, there were some non-structural cracks visible along the corridor and on the verandah side such as shown in Figure 6. Also the collapsed boundary wall in the side of the building can be seen in Figure 6, where a photograph taken from the entrance shows the cracking of paved surface. Some ground settlement can also be noted.

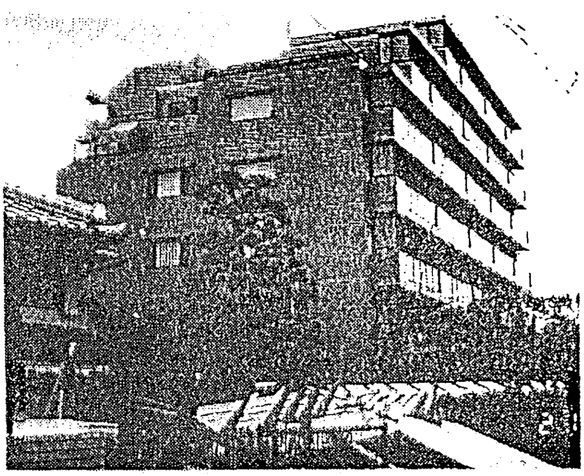

Figure 5: View of Seemingly Undamaged Building

No large scale lateral ground displacement was noticeable in the immediate vicinity of the building. However, some small sand blows were visible indicating the occurrence of limited soil liquefaction phenomenon. The maximum undulation in the existing ground around the building was up to about 100mm. To ensure that the building was as unscathed as it appeared to be, it was decided to make detailed measurements of the tilt and settlement of the building. The result was contrary to the speculation based on the casual observation.

Figure 6: Cracking of Paved Surface, Collapse of Boundary Wall and Cracking of Non-structural Wall Observed after the Earthquake

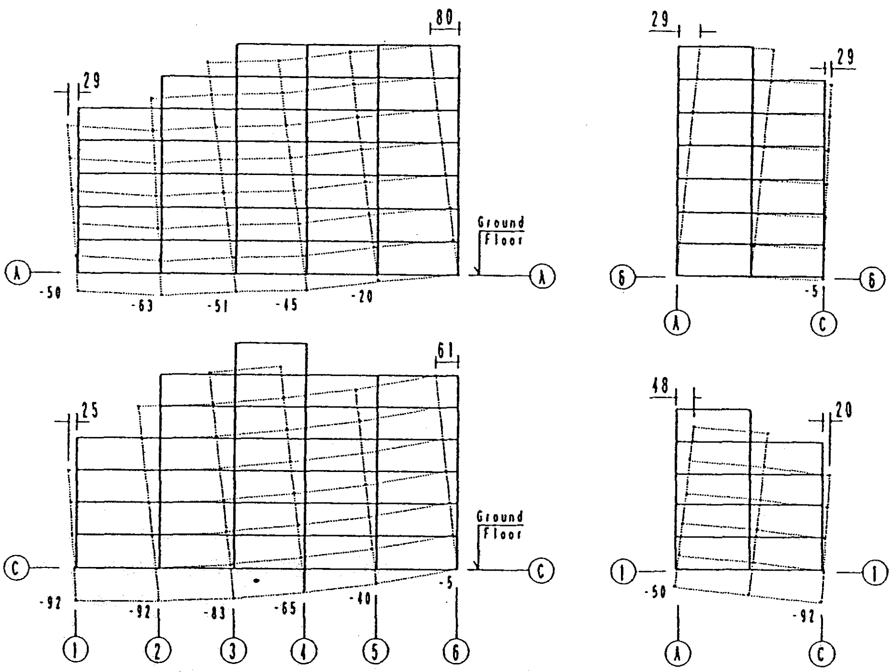

Figure 7: Illustration of the Nature of Differential Settlement and Tilting of the Building Based on the Detailed Measurement and Investigation.

Detailed measurements were made subsequently. The location map of boreholes in Figure 8 shows the temporary bench mark (TBM), relative to which settlements were measured. It was revealed that there was a differential settlement of about 90mm between eastern and western ends of the building, as seen in Figure 7. Note that the grid-line numbers given in Figure 7 correspond to those shown in the foundation plan in Figure 3. It was found that the south-east corner (A-6 in Figure 3) of the roof was out of the plumb-line by about 80mm as seen in Figure 7. It was inferred that such significant differential settlement and tilting of the building meant that the pile foundation supporting the structure must have incurred substantial damage. Skeletal frame analysis was carried out to evaluate the forces and moments exerted in different members due to the measured settlement and tilting of the building superstructure. This provided with a grasp of the existing state of stresses in the building, which was vital in the process of selecting the appropriate approach to restoration work.

4 Damage to Pile Foundation

To check the extent of damage to the piles, excavation was carried out from the outside of the building to expose the top of piles under one of the footings. As anticipated, the top of the exposed PHC piles were found to have sheared and crushed at the underside of footing. Evidently most of the precast piles supporting the structure were totally broken at the top. Under the situation, most of the building weight appeared to be supported by the underside of footings and foundation beams by direct bearing, serving as some sort of spread foundation. With the piles designed to transfer the load rendered ineffectual, it was clear that the building was vulnerable to progressive distress. The situation was made specially dreadful because of the impending danger of large aftershocks following the main earthquake event.

A prompt action towards immediate measures for prevention of further deterioration and destabilization was was urgently needed, before initiating detailed studies required for developing appropriate strategy for restoration. As an immediate precautionary measure, cement grout was injected around all the column footings so as to stabilize the surrounding ground.

5 Soil Profile of the Site

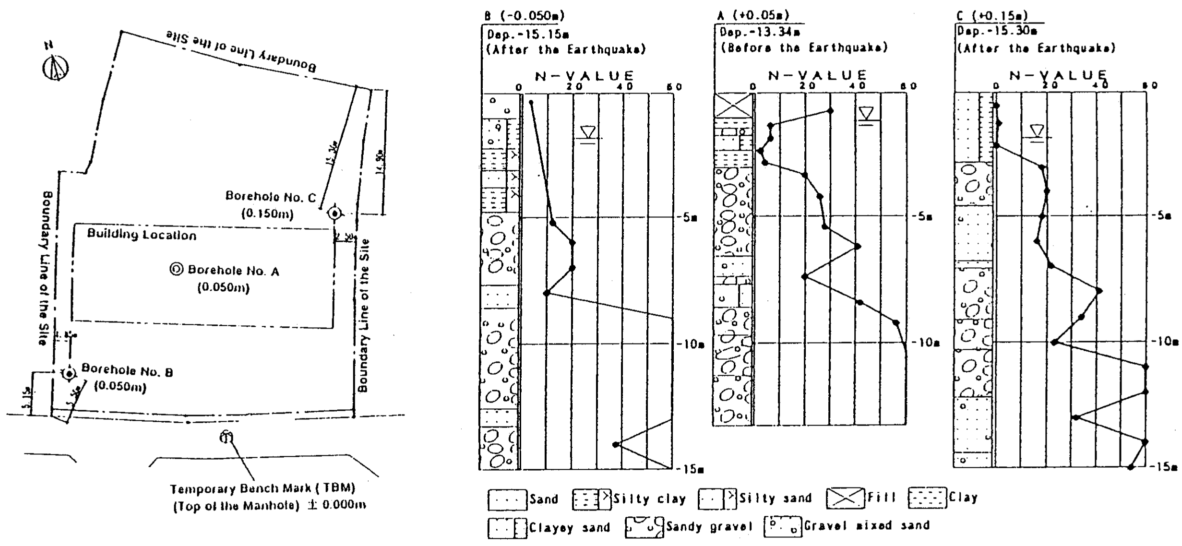

Figure 8: Location of Boreholes and Soil Profiles. (Borehole A is from before the building was constructed, and boreholes B & C were drilled after the earthquake).

The boring for soil exploration carried out for the purpose of the pile foundation design was located close to the center of the site as shown by borehole A in Figure 8. The site condition is fairly stiff as can be noted from the standard penetration test N-value profiles. The depth to underground water was about 1.0m from the ground level. Additional exploration borings were carried out after the event to see if there were any significant changes in the ground conditions because of the earthquake. The additional borings are indicated by B and C in Figure 8. It can be noted that the three borings together form a section through the site, and the soil profile broadly consists of alternate layers of sandy soil and sandy gravel of varying thickness. Overall, the soil profile can be considered to consist of 3 layers. There is surface fill of 1 to 2.8m at the top, with the thickness increasing from A to C. Below this and up to a depth of GL−7.2 to −8.5m is the alluvium underlain by diluvial deposits, with fairly high N-values.

The underground water was at about GL−1.1m in exploration before the earthquake, while it is at about GL−1.8m based on exploration after the earthquake. The difference may be attributed to seasonal variation. From practical point of view, the ground profiles from the three exploration borings can be considered to be quite consistent.

6 Directives for Restoration Project

With the immediate precautionary measure in place, restoration targets and directives for necessary further studies needed for planning and execution of restoration measures were defined with a view to ensure adequate safety of the building in the long run. The following directives were formulated for the development of a detailed plan to repair and strengthen the building foundation system.

1. The tilted building superstructure would be jacked up to the vertical plumb line.
2. The top portion of the piles and connections with the footings would be repaired and strengthened to a level well above the initial design strength.
3. The building weight will be supported by a suitable underpinning method before cutting off the piles from the footing for restoration and strengthening.
4. As the underground water level is quite high at the site, adequate earth retailing system would be designed and installed to prevent cave-in collapse of the ground during excavation to expose top portions of the piles.
5. Utmost care would be taken to select the construction method and jacking technique, such that the disturbance and distress to the already weakened building structure would me minimum.

7 Selection of Foundation Restoration Method

A meeting of a group of experts was convened to discuss the restoration project. The purpose of the discussion was to develop a list of possible construction and underpinning methods that may be applicable under the given situation. The critical aspect of the restoration project was the appropriate system for supporting (underpinning) the building while it is being jacked back up to plumb and level, after which the damaged piles could be repaired and strengthened. Of the several ideas for temporary support that were brought up during the group discussion sessions, five were considered to be viable for construction under the situation. These were:

I. Caissons under shear walls.
II. Spread footing under shear walls.
III. Steel pipe piles under shear walls.
IV. Steel pipe piles under footings.
V. Fill-in wall all around in the first floor.

The five different options were then evaluated in detail to compare their relative merits and demerits. The process actually involved is shown in the flow diagram in Figure 2. The comparison for relative merits and demerits of different methods involved visualization of the sequence of steps involved in each method and evaluation of the possible relaxation in bearing support during the construction process. Skeletal stress analysis based on the expected disturbance to the structure at different stages of construction was carried out to evaluate the extent of distress to the structure.

The primary selection criteria was that the possible effects to the structure due to the construction method was within tolerable limits. Based on this, methods I, II and IV were considered to be acceptable. Next, of course, was the cost criteria. Since the first two of the three methods involved dismantling the ground floor slab for the construction work, using steel pipe piles under the footings (method IV) was found to most cost-effective. Hence, this method of temporary support (underpinning) was selected for the restoration work.

A comprehensive restoration plan was developed based on the use of steel pipe piles for temporary support and the plan was successfully implemented. A typical scenario of changes in the share of a column load of about 4.0MN coming to a footing taken by different elements at different stages of construction is depicted in Figure 9. As can be noted, initially most of the footing load is taken by direct bearing of footing and foundation beams, and only a small part is borne by the damaged pile. There is a slight increase in the share of load taken by foundation beam and piles at stage 2 compared to that existing initially. However, this is of the order of only 10%, and was considered to be of not much concern.

Stages in the installation of steel pipe piles for underpinning of footing with three piles:

1. Initial (before excavation)
2. Footing underside is partly excavated
3. First steel pipe pile (SPP) installed
4. Excavation for second SPP
5. Second SPP is installed
6. Footing is completely excavated
7. Case of no load taken by concrete piles
8. Third SPP installed

Total = Sum of load taken by footing, foundation beam & damaged concrete pile.

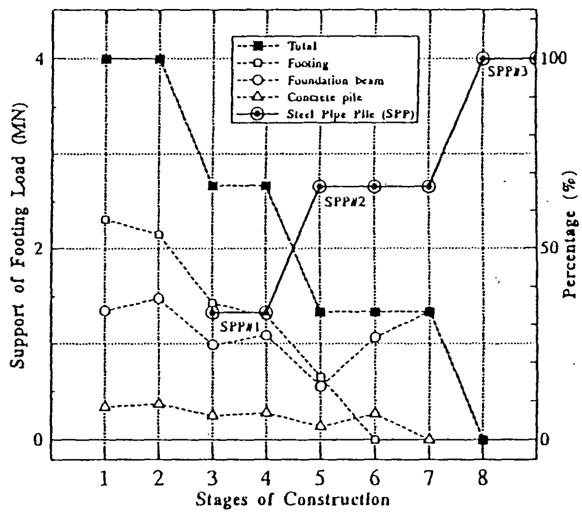

Figure 9: Load Sharing Pattern During the Installation of Steel Pipe Piles

Obviously there was a constant concern regarding the impending danger from possible aftershocks during the time restoration was in progress. The concern arose from one unavoidable deficiency in the restoration plan, which was the lack of equitable provision for horizontal resistance of the structure. Considering the limited choice under the circumstances, there was essentially some risk involved in this respect. That there were no aftershocks closer in size to the main event may have been an element of luck favoring successful implementation of the restoration plan.

8 Construction Sequence for Temporary Support

The installation sequence of the temporary support system is illustrated diagramectically in Figure 10. Circular liner sheet for earth retaining was utilized to excavate around the

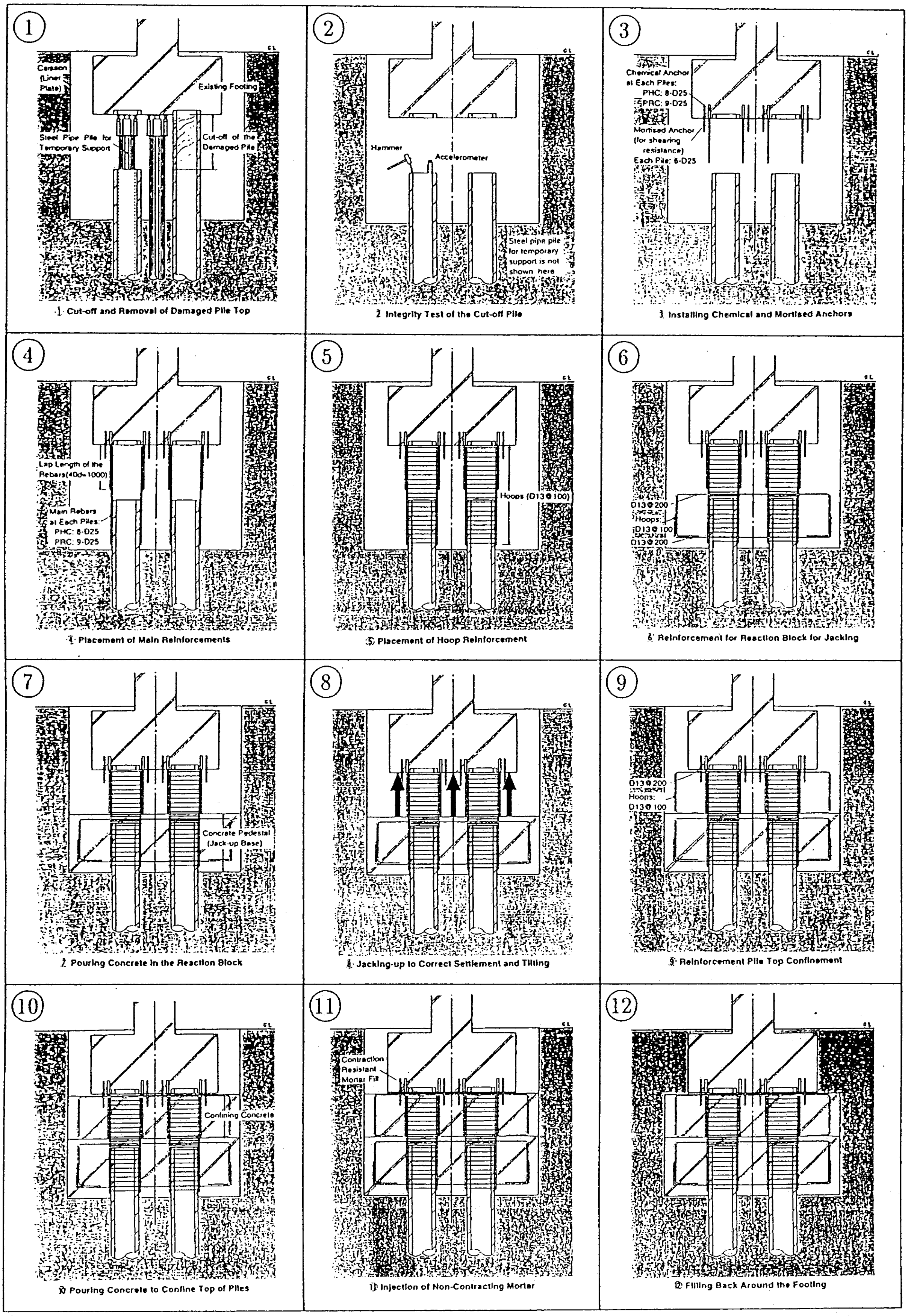

Figure 10: Diagram Illustrating Typical Underpinning and Repair Sequence for Restoration. (Note: To avoid obscurity, subsequent steps do not show steel pipe piles.)

footing to expose top of piles to a depth of about 3.5m below the existing ground level. Then the steel pipe piles were installed statically by jacking against the underside of footing. Altogether 39 steel pipe piles (26 ϕ355.6×9.5mm thick and 13 ϕ318.5×7.9mm thick) were installed. The bearing capacity was directly measured as the piles were being installed. The maximum load for installation was considered to be 2.0MN. for larger diameter steel piles and 1.0MN for smaller ones. The layout of the steel pipe piles is shown in Figure 3 above.

After a steel pipe pile was installed to have the required bearing capacity, part of the load from the footing was transferred to it and the damaged portion of the precast concrete pile was cut-off and removed as shown in step 1 in Figure 10. The nature of damage of all the piles were recorded, and is discussed in the following section. The lower part of the cut-off pile was tested for integrity as seen in step 2 of Figure 10.

For connection of the cut-off piles to the footing, chemical anchors for main reinforcement and mortised anchors for shear connection were installed (step 3) at the underside of the footings. As seen in step 4, the main reinforcement for the connection were placed to have an appropriate lap-length with the chemical anchors. Then the hoop reinforcements were placed as shown in step 5 in Figure 10. After that, the reinforcement for the concrete reaction block (pedestal) were placed (step 6) and concrete was placed (step 7). Note that the steel pipe piles actually taking the footing load are not shown in subsequent steps in Figure 10 for clarity of diagrams.

Installation was carried out at all the footings up to step 7, so that the simultaneous jacking-up operation described below could be undertaken. It was found that all the piles supporting the building had failed except for the six numbered 29 to 34 in Figure 3, where the steel pipe piles can be noted to be absent in footings 6A, 6B and 6C. Once the building structure was jacked up (step 8 in Figure 10) against the reaction blocks to correct for differential settlement and tilting, further reinforcements were placed on top of the reaction block (step 9) and concrete was poured (step 10) to confine the top of piles. The jacking operation is discussed in more detail below. Finally, non-compressible mortar was injected below the footing (step 11) so as to transfer the footing load to the confining concrete block below, and the excavation was filled back (step 12).

9 Failure Pattern of the Piles

As mentioned above, the six piles along grid-line 6 on the east side of the building were found to be intact. It can be noted from Figure 7 that the settlement along this grid-line measured relative to the TBM in Figure 8 is also quite small. Figure 11 shows the top of two piles under footing C6, which are numbered 29 and 30 in Figure 3. It can be noted that the piles are undamaged. The piles under the other two footings in grid-line 6 were similarly intact.

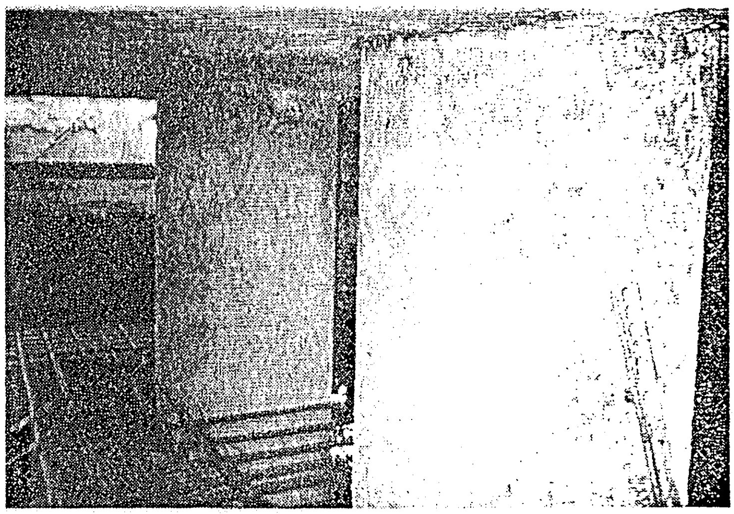

Figure 11: Example of Undamaged Pile

Aside from the six, all the other piles had failed in some sort of a combination of shearing and compression mode. The conditions of a PRC pile numbered 4 in Figure 3, is shown in Figure 12. Photographs taken from four mutually perpendicular sides (denoted as A, B, C and D sides) together with rough sketches of the damage size in each case are shown for pile 4. Similarly, Figure 13 shows by photographs and sketches the damage to a PHC pile, which is numbered 25 in Figure 3.

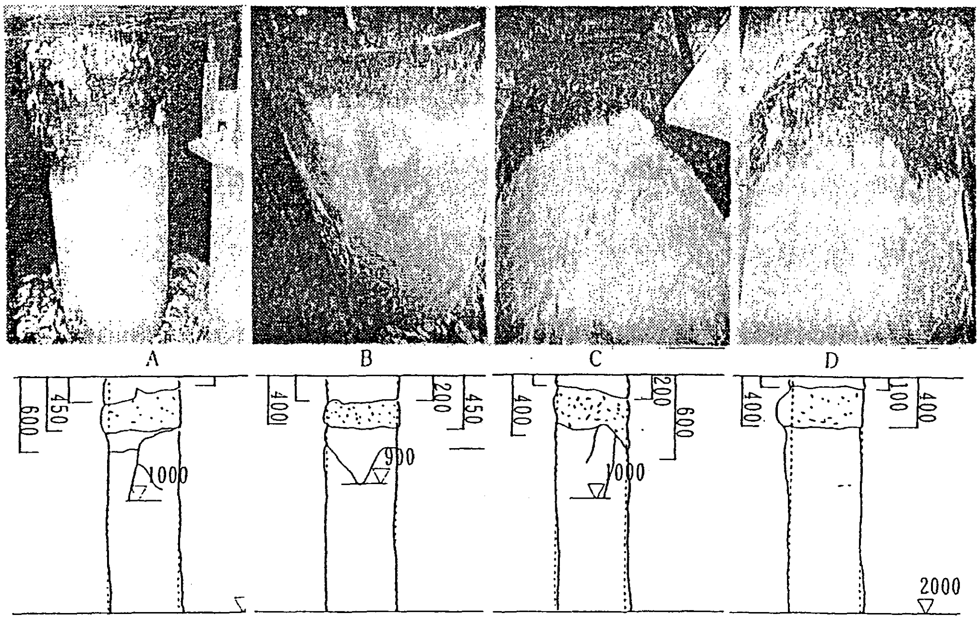

Figure 12: Photographs and Sketches of the Four Mutually Perpendicular Sides of the Top of a Typical damaged PRC pile

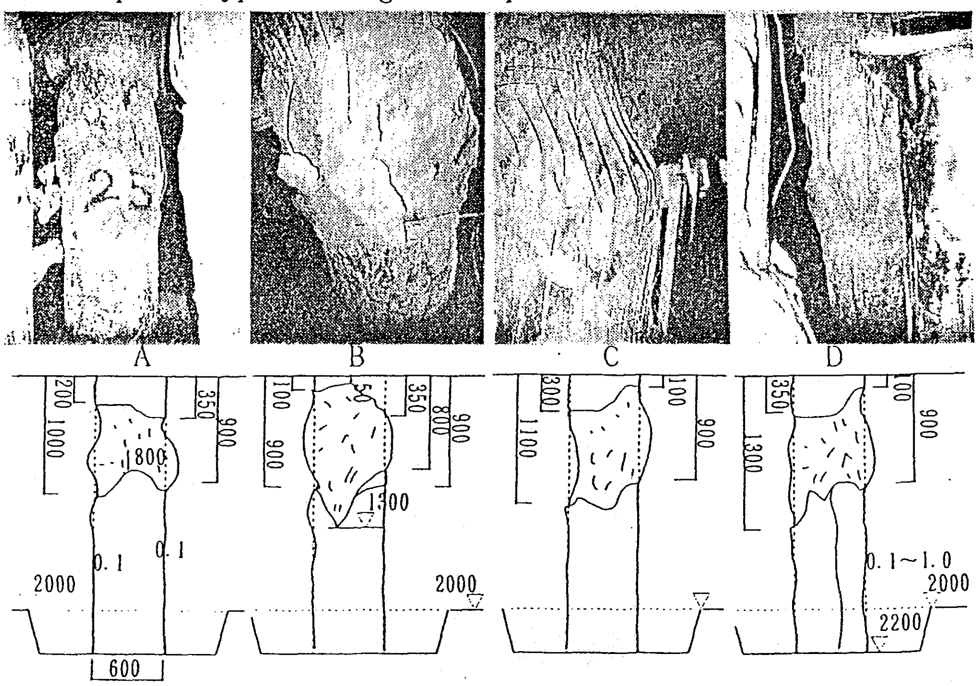

Figure 13: Photographs and Sketches of the Four Mutually Perpendicular Sides of the Top of a Typical damaged PHC pile

It may be noted that the PRC pile of Figure 12 is located on the side of the building where the settlement is largest, while the PHC pile of Figure 13 is located next to the grid-line of minimum settlement (eastern end of building). However, the damage in the PHC pile can be noted to be much more severe compared to that in the PRC pile. The reason most likely lies in the relatively higher ductility of the PRC pile. Brittle nature of the failure initiated by shearing crack and subsequent crushing in compression was noted in case of all the PHC piles.

10 The Jacking-up Operation

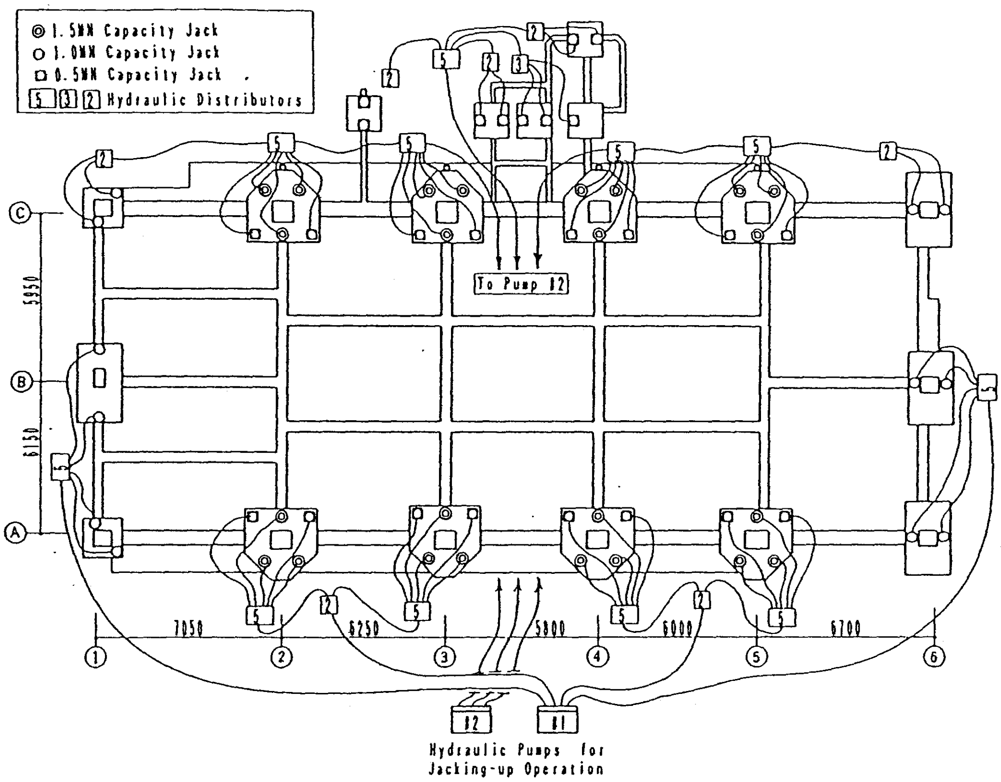

Figure 14: Plan Showing the Hydraulic Layout System and the Location Jacks Utilized in the Jacking-up Operation.

The jacking-up process involved the use of a hydraulic distribution system simultaneously operating 61 hydraulic jacks. The layout plan of the hydraulic jacks of capacities 1.5MN, 1.0MN and 0.5MN (numbering 18, 34 and 9 respectively) are shown in Figure 14, where the hydraulic jacks of different capacities are represented by different notations. Two pumps were utilized to operate the jacks through a hydraulic distribution system shown in Figure 14. A typical situation of jacking-up operation can be seen in the photograph of Figure 15.

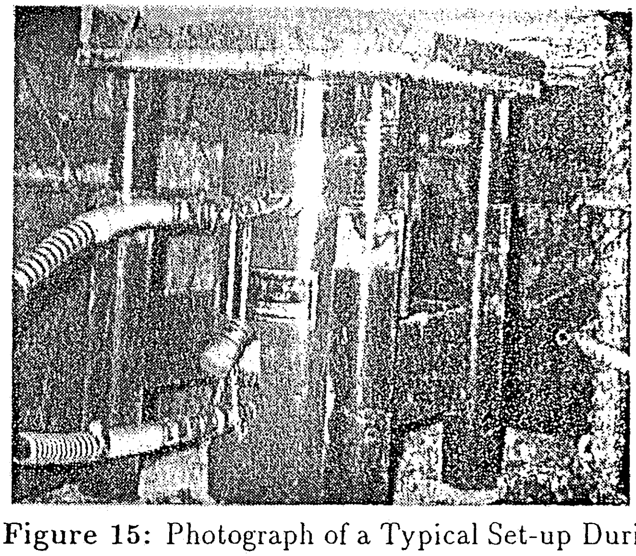

Figure 15: Photograph of a Typical Set-up During the Jacking-up Operation.

The jacking-up operation was carried out in small steps with close monitoring of level at all the footings, as well as the extent of recovery of the tilt of the building. There were altogether 18 steps. Figure 16 shows the extent of the recovery of the differential settlement of the building along grid-lines A-A (south) and C-C (north) at selected steps of jacking-up operation. The dotted line shows the existing settlement, and the differential settlement can be seen to decrease steadily as the jacking-up operation continued. After the end of the final step, the differential settlement can be noted to be practically negligible.

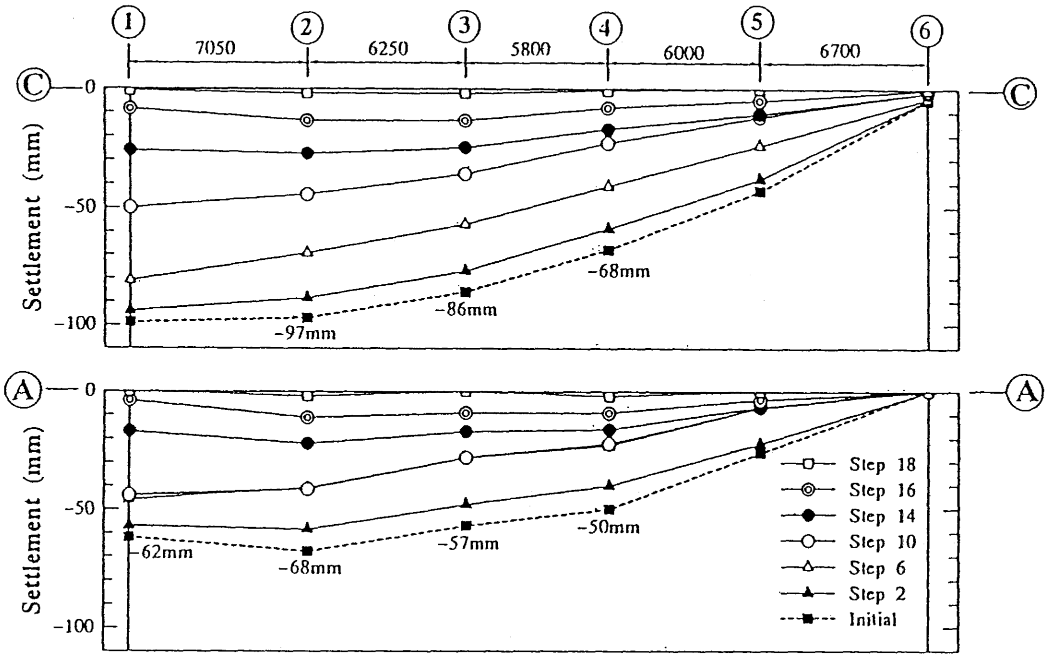

Figure 16: Reduction in the Settlement of the Building at Different Stages of the Jacking-up Operation

11 Cracks in the Building

After the building was jacked-up to its original level and upright position and after completion of the concreting and back filling of all the footings, the superstructure of the building was inspected in detail. No sigh of any distress was found in the structure. Consistent with the correction in tilting and differential settlement, the yawning non-structural cracks, such as seen in Figure 6 had been closed to a large extent. The cracks were further treated by injecting binding material. The type and method of grouting the cracks varied depending on the size and location of the cracks. After the cracks were repaired, final finishing was done.

12 Closure

It is imperative that common residential buildings may suffer some structural damage under extreme earthquake events. However, it is expected that rational design can prevent total collapse by judicious provision of energy absorption mechanism in the structural system. Rehabilitation and restoration of structures partially damaged in extreme events is of vital importance in earthquake prone areas like Japan and California. Intentional or not, the new building at Nishinomiya happened to develop the energy absorption mechanism through failure of the top of piles. The restoration plan that involved appropriate underpinning and simultaneous operation of 61 hydraulic jacks was successfully implemented to jack-up and repair the building. The differential settlement and tilting were practically nil when the restoration was complete. There was an element of luck that no large aftershocks occurred during the implementation. It is anybody's guess what may have happened had there been a major aftershock on the day the jacking-up operation was being carried out. The experience from the project is believed to be valuable for restoration and reconstruction of pile foundations damaged by extreme earthquake events.

Reference

KOBE University, 1995. 2nd Report on Great Hanshin Earthquake. (Collab. Hyogo Prefectural Govt., Kobe City Govt., and Constr. Engr. Res. Inst. Foundation).