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Real-Time Monitoring for Monolithic Movement of a Heritage Curtilage Using Wireless Sensor Networks

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Abstract: Since monolithic movement is considered a promising technology to relocate historical buildings, corresponding real-time monitoring is of great interest due to the buildings' age and poor structural integrity. However, the related paperwork and practical applications are still limited. This paper describes a wireless sensor network (WSN)-based strategy as a non-invasive approach to monitor heritage curtilage during monolithic movement. The collected data show that the inclination of the curtilage is almost negligible. With the aid of finite element simulation, it was found that the crack displacement curves changed from −0.02 to 0.07 mm, which is affected by moving direction while the value is not enough to cause structural cracks. The deformation of the steel underpinning beam, which is used to reinforce masonry walls and wooden pillars, is obviously related to the stiffness in different directions. Additionally, the strain variations of the steel chassis, which bear the vertical loads from wooden pillars and masonry walls, are less than 0.04%. This indicates that they are kept within the elastic range during monolithic movement. This work has proved that the WSN-based approach has the potential to be applied as an effective route in real-time monitoring of the monolithic movement of an historic building.

Keywords: wireless sensor network; historic building; real-time monitoring

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1. Introduction

Building monolithic movement, a kind of procedure to relocate the position or direction of an existing building whilst maintaining its integrity and availability, has attracted much interest due to its resource-saving, environmentally friendly and low-cost advantages [1]. Since 1998, a series of related protection statutes and regulations with an emphasis on the protection of preserved historical buildings have been published in mainland China, aiming at materializing historical influences and differences [2]. The corresponding monolithic movement technology is also considered as a new resolution to conserve historical buildings whilst adapting them to new conditions and uses [3].

Compared with traditional demolition and reconstruction, the monolithic movement of historical buildings has the particular advantage of maintaining the original humanistic value and the overall structural integrity. The embryonic form of modern movement-engineering technology can be traced back to 1983, when a school building with a masonry structure (weighing 8000 kN) was moved a distance of 15 m in the city of Warrington, England [4]. With the advancement in technology, the monolithic movement techniques of historical buildings are being developed and implemented worldwide [5]. For example, Kossakowski et al. [6] described the case of the relocation of the Rogatka Grochowska building, which was carried out in Warsaw. The related work and projects in China came late to developed countries. Shan et al. [7] implemented a complex monolithic movement of the Ci-yuan temple with a brick-wooden structure, which was built in the Tang Dynasty and was located in Anyang City in China. Moreover, the documented application projects, including Jinlun Guild Hall (Qing Dynasty), Shanghai Concert Hall (1930), and Centennial Minli high school of Shanghai (1920), were also realized through historic building moving [8,9]. The existing cases signify that three general routes, including moving with rolling bars, moving with a slide layer, and moving by trailer transportation, have been performed successfully as monolithic movement applications of historical buildings in China. To date, due to the buildings' intrinsic rarity and structural aging, only limited paperwork has been disclosed, and few practical applications of the monolithic movement technology of an overall heritage curtilage courtyard have been carried out; however, this technology is slowly becoming more widespread and pervasive in modern civil infrastructure.

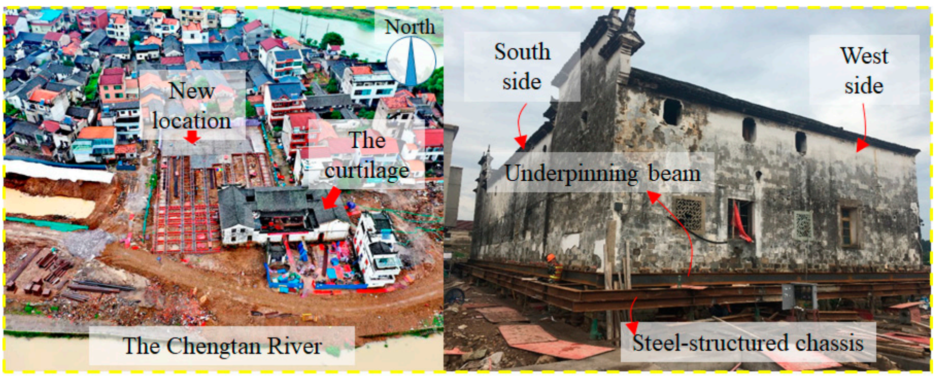
During the monolithic movement procedure, it is well known that prioritizing the protection of the historical building, with its intrinsic cracks and cavities, against destructive loads is essential. A program of non-invasive monitoring needs to be undertaken to inform about the ongoing structural status [10–12]. As the moving of historical building means there are some risk factors, the traditional inspection methods for structural cracks, deterioration, and damage tend to be inconvenient and dangerous. With the progress of wireless communication and sensor miniaturization, the wireless sensor network (WSN) system—with a series of smart sensors in a self-organizing and multi-hop manner for monitoring structural deformation—has been developed in recent years [13,14]. Compared with a customary wired network, the WSN boasts easy deployment, a dramatically lower cost of installation and maintenance, and it provides a flexible and manageable approach to monitoring remotely in real time [15,16]. Dong et al. [17] compared the WSN system with the wired sensing system for the performance of a 2-story, 2-bay concrete frame building, and they found that the WSN achieved the same quality of data as that of the wired system. Furthermore, the existing research has proved that the WSN-based approaches could identify the existence and location of damage for long- and short-term monitoring to achieve structural health and safety assessments of historic buildings whilst maintaining their structural integrity and functionalities [18,19]. For example, Wu et al. [20] introduced a dedicated WSN into Torre Aquila, built in the 13th century and located in the city of Trento, Italy, to evaluate its static and dynamic state by utilizing accelerometers and deformation sensors. They also found that the collected data are in agreement with the prediction from the three-dimensional finite element (FE) results. Samuels et al. [21] developed a WSN for monitoring the tilt in the walls of the St. Paul Lutheran church, an historic masonry church with a timber-framed roof, under rehabilitation. Potenza et al. [22] undertook the deployment, test, and management of a WSN for the structural monitoring of the Basilica S. Maria di Collemaggio with masonry structure, which is designed for seismic and dynamic response analyses via acceleration, crack opening, and wall inclination measurements. Mesquita et al. [23] adopted temperature, relative humidity, and displacement sensors to perform a one-year monitoring of the Foz Côa Church (in Portugal), a damaged historic structure from the 16th century based on the WSN system. Barsocchi et al. [24] presented an application of the WSN technology on the Matilde Tower in Livorno (Italy), an historic masonry tower built in the Livorno harbor, to monitor the structural health over the long term and detect potential damages in real time. However, given the uniqueness and the preservation of each historical building, real-time monitoring applications based on the WSN system for the case of its monolithic movement are still very challenging, and few related works have yet been documented.

Therefore, this paper aims to explore the real-time monitoring ability of the WSN in a monolithic movement project for a heritage curtilage with a masonry–timber structure. Considering the age and the poor structural integrity of the heritage curtilage, four kinds of smart sensors were adopted with the aid of practical engineering experiences and FE simulation to real-time monitor the structural deformation and deterioration. Then, the acquisition system receives the processed data and transmits them to a cloud platform via wireless remote communication (3G/4G/GPRS). Finally, all the data can be accessed directly at the preferred time with wireless communication in locations where internet access is available. This paper not only develops a comprehensive scheme for monolithic movement monitoring of a heritage curtilage, but also provides an in-depth understanding of the structural deformation and deterioration during monolithic movement.

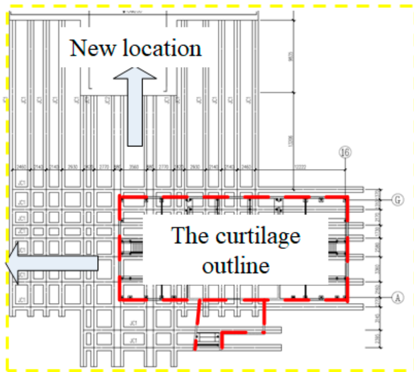
2. Methods

2.1. Case Study

The south-facing curtilage (Figure 1a) was a traditional courtyard with two-story masonry–timber structure, which was located in Shengzhou City, Zhejiang Province, China, with cover an area of 685.23 m^2, ridge height of 7.45 m, and gross weight of 3.65 $\times$ 10^6 kN. According to the architectural style, it was identified as being built in the late Qing Dynasty or the Republic of China. The combination of beam-lifted and Chuan-Dou frame was adopted as the load-bearing structure, and there were stone foundations and capstones under the wooden columns. The primary and secondary entrances and exits of the inner courtyard were both located in the south, the surrounding exterior walls were built with a rowlock cavity wall, and the wall foundation was stacked with rock blocks. In coordination with the emergency project of flood prevention improvements of Chengtan River within Cangyan village, the curtilage movement was planned to be conducted over a one-week period and included as follows: monolithic moving 15 m in the western direction over four days, and then 40 m in the southern direction to the new location in three days, shown in Figure 1b.



(a)



(b)

Figure 1. (**a**) The aerial view of the curtilage and its new location; (**b**) the schematic diagram of the monolithic moving procedure.

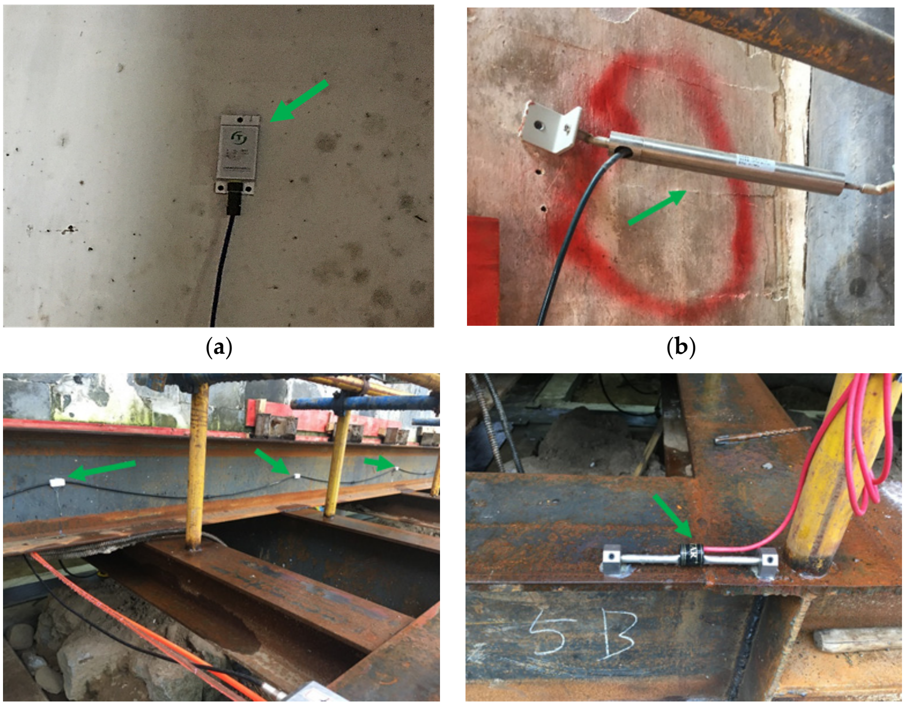
The monolithic movement of the heritage curtilage was achieved based on lifting jack technology, and the speed was set as 0.8–1.6 mm/s [3] under a horizontal pushing load provided by the push-in jack (SCLRG-100-500-T) with a rated pushing load of 1000 kN and a pushing distance of 500 mm, as shown in Figure 2a. Due to the complicated structural form and the weak structural integrity, the bottom of the masonry wall and wooden pillar were reinforced by a two-clip steel beam with block stone fillers between them to form underpinning beams. In addition, 9 longitudinal and 16 transverse H-shaped steel beams were assembled in Figure S1 (Supplementary Materials) as the integral, and the underpinning chassis was built in a grid pattern with appropriate diagonal crossing beams to enhance the horizontal stiffness The protective supporting platform was also adopted for the superstructure of the curtilage. Thereby, the vertical loads were transferred from wooden pillars and masonry walls to the steel structural chassis, and the top major structure was separated from the original foundation to form a mobile body, which was displayed in Figure 2b. The monolithic movement was achieved by the sliding movement between upper and bottom rail beams using the floating jack as the special sliding support, which could reduce the horizontal resistance and regulate vertical deformation. The upper and bottom rail beams were connected to the steel chassis and were placed on the foundation, respectively, shown in Figure 2b.

(**a**) (**b**)

Figure 2. The photos of (**a**) the push-in moving process and (**b**) the mobile body.

2.2. Monolithic Movement Monitoring Scheme Based on the WSN System

The proposed WSN formed a data collection network of sensors, which were driven by the power supply to serve a specific target-oriented application. The real-time monitoring for the monolithic movement of the heritage curtilage consisted of the parameters of measurements such as cracks, strain, and deformation, thereby obtaining the structural response. Figure 3 presents the four kinds of sensor nodes used in this paper: (1) inclinometer (XKKJQJ-77, ±15°, 0.01°), showing the inclinations of the timber column and masonry wall; (2) displacement transducer (XKKJZX-2109, 0–20 mm, 0.1%F.S/0.5%F.S), monitoring the crack behaviors of the masonry walls and the masonry–timber connections; (3) series inclinometer (XKKJWY-5675, ±30°, 0.015°), to reveal the deformation of the steel beam at the bottom of the masonry wall; (4) strain gauge (XKKJZX-212, 3000 με, ≤0.5%F.S.), attached to the steel chassis to obtain their strain variations. It is noted that the first item in the above brackets is the sensor type, the second is the measuring range, and the third is the precision.



(**a**) (**b**)

(**c**) (**d**)

Figure 3. Four kinds of sensor nodes used: (**a**) inclinometer, (**b**) displacement transducer, (**c**) series inclinometer, and (**d**) strain gauge.

Figure 4 demonstrates the WSN system schematic to collect and observe the monitoring data in real-time and remotely. Taking displacement transducers as an example for simplicity in Figure 4a, the sensor nodes were deployed on the primary locations. The corresponding acquisition devices received processed data from the sensor nodes and transmitted these data to a cloud platform within the monitoring server (Figure 4b) via wireless remote communication (3G/4G/GPRS). Then, all the data could be accessed directly at preferred times with a personal computer (Figure 4c), mobile phone, or other wireless communication in locations where internet access is available.

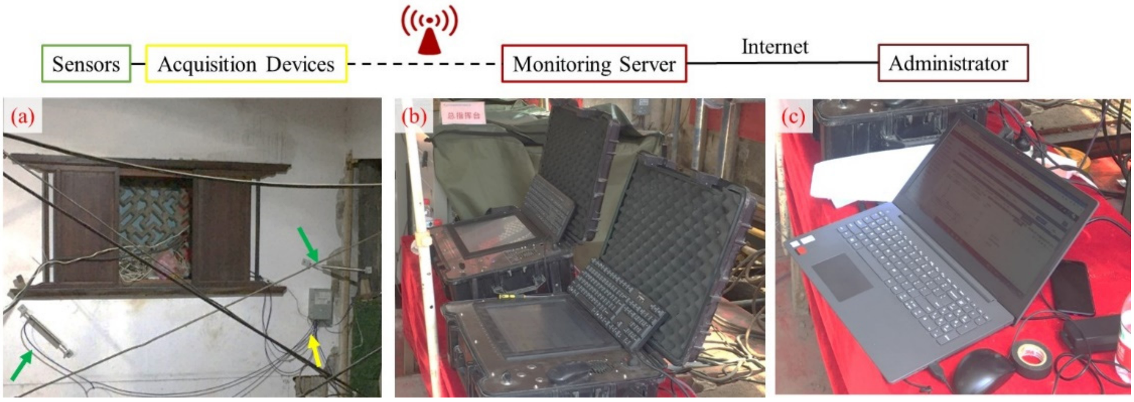


Figure 4. The WSN system schematic: (**a**) sensors (pointed with yellow arrows) and the corresponding acquisition devices (pointed with green arrows); (**b**) monitoring server; (**c**) wireless communication.

2.3. Numerical Method

In order to understand the structural behaviors and obtain the rational distributions of the displacement transducers, three-dimensional FE models of the exterior walls were built in Abaqus software, consisting of masonry assembly, window frame made of rock, steel reinforcing beams, and block stone fillers (shown in Figure 5). Due to the same geometric structure of the east and west walls, only the west wall was simulated in this paper; half of the north wall was built considering its symmetric geometric structure. Based on computational efficiency, the masonry assembly was regarded as an isotropic composite material, and it followed a nonlinear elastic–plastic constitutive relation [25], which is described by the embedded Concrete Smeared Cracking (CSC) model in Abaqus. The used CSC model, detailed by Lotfi and Shing [26,27], adopted a $J2$-plasticity model with nonlinear isotropic strain hardening and softening to demonstrate the mechanical behaviors of uncracked materials, and a nonlinear orthotropic model to describe the behaviors of cracked materials. The top structure of the studied FEA model was fully fixed, and an incremental displacement loading was applied at the bottom to simulate the deformation during monolithic moving. The basic mechanical parameters of the exterior walls are listed in Table S1.

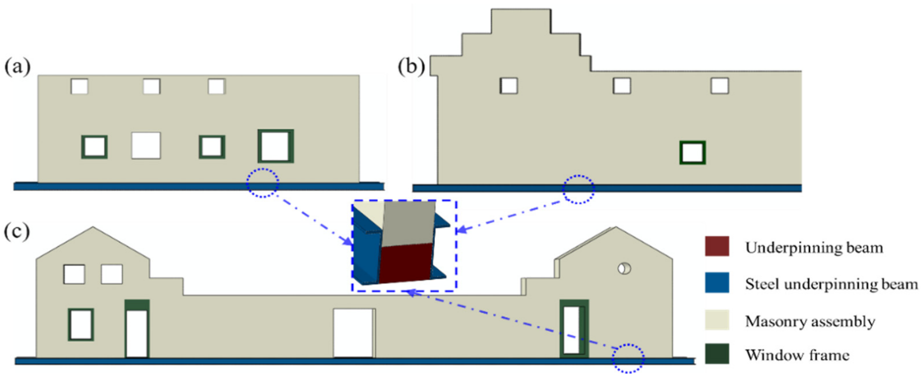


Figure 5. FE models of exterior walls: (**a**) west side, (**b**) north side, and (**c**) south side.

3. Results and Discussion

3.1. Inclination

The near-zero data of the angular variations, collected from all inclinometer sensors, can be found in this case, which indicate that the inclination of the timber column and masonry wall was negligibly small during the complete monolithic movement process. Figure 6 takes the W01 and W02 inclinometers as the typical examples, which are installed on the west wall at 1.5 m above the first-floor ground. Each anchorage point could provide two datasets along east–west (E–W) and south–north (S–N) directions, respectively. Whether moving to the west or north, the inclination angle changed from approximately −0.0025 to 0.0025 degree in both the E–W and S–N directions. It can be deduced that the monolithic movement of the heritage curtilage is expected to realize the aspect of almost negligible inclination.

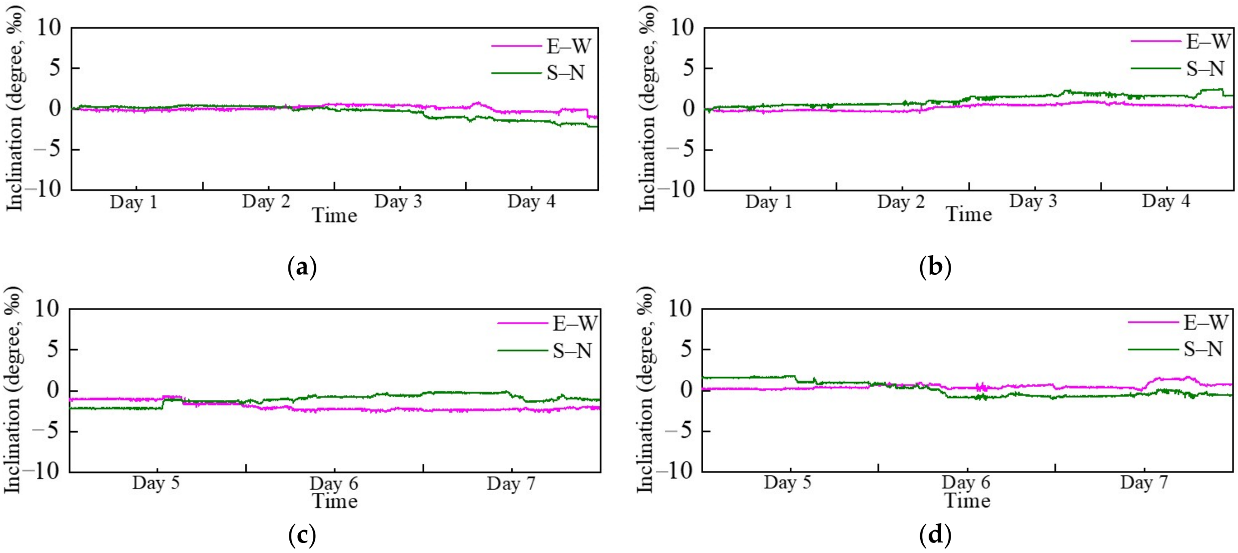


Figure 6. The obtained inclination angle: (**a**) W01 and (**b**) W02 during westward movement; (**c**) W01 and (**d**) W02 during northward movement.

3.2. Crack Behaviors

As already highlighted, a numerical simulation based on the CSC model in Figure 7 is performed to predict the crack locations on the exterior walls, in which the smeared crack is studied instead of an individual crack. Figure 7a–c displays the equivalent plastic strain (PEEQ) distributions of the exterior walls to reveal the crack locations most easily; thereby obtaining the corners of the window and door that tend to crack under a relative deformation between the upper and lower parts of the walls. Accordingly, just four displacement transducers, denoted as E01, S01, W01, and N01 in Figure 7d, were attached to the exterior walls on the east, south, west, and north sides, respectively. Moreover, masonry-timber connections were also taken into consideration due to the masonry-timber structure of the curtilage. As illustrated in Figure 8d, the displacement transducers at the primary entrance (A01) and on the interior wall (A02) were taken as typical examples in this case.

Figure 8 shows the crack displacement variations along with the moving procedure. For the exterior walls, the crack displacements of the S01 and N01 sensors are slightly higher than that of the E01 and W01 sensors when moving west in Figure 8a, which means that the south and north walls are easier to crack under the horizontal west-pushing loads. When moving north, the east and west walls become easier to crack, as deduced from Figure 8b. As for the masonry-timber connections, the crack displacement variations of the A01 and A02 sensors are also plotted, which shows that the changing range of the A01 curve is larger than A02 when moving west; when moving north, the A02 curve changes more obviously. Moreover, the FE results based on the CSC model also show that when the displacement variations within the areas of the displacement transducers are below 0.17 mm, there is no plastic deformation that can be observed. It is worth noting that all of the crack displacement curves are changed in the range of −0.02 to 0.07 mm in Figure 8, which indicated that the crack displacement variations are very small and insufficient to give rise to structural cracks during the monolithic movement.

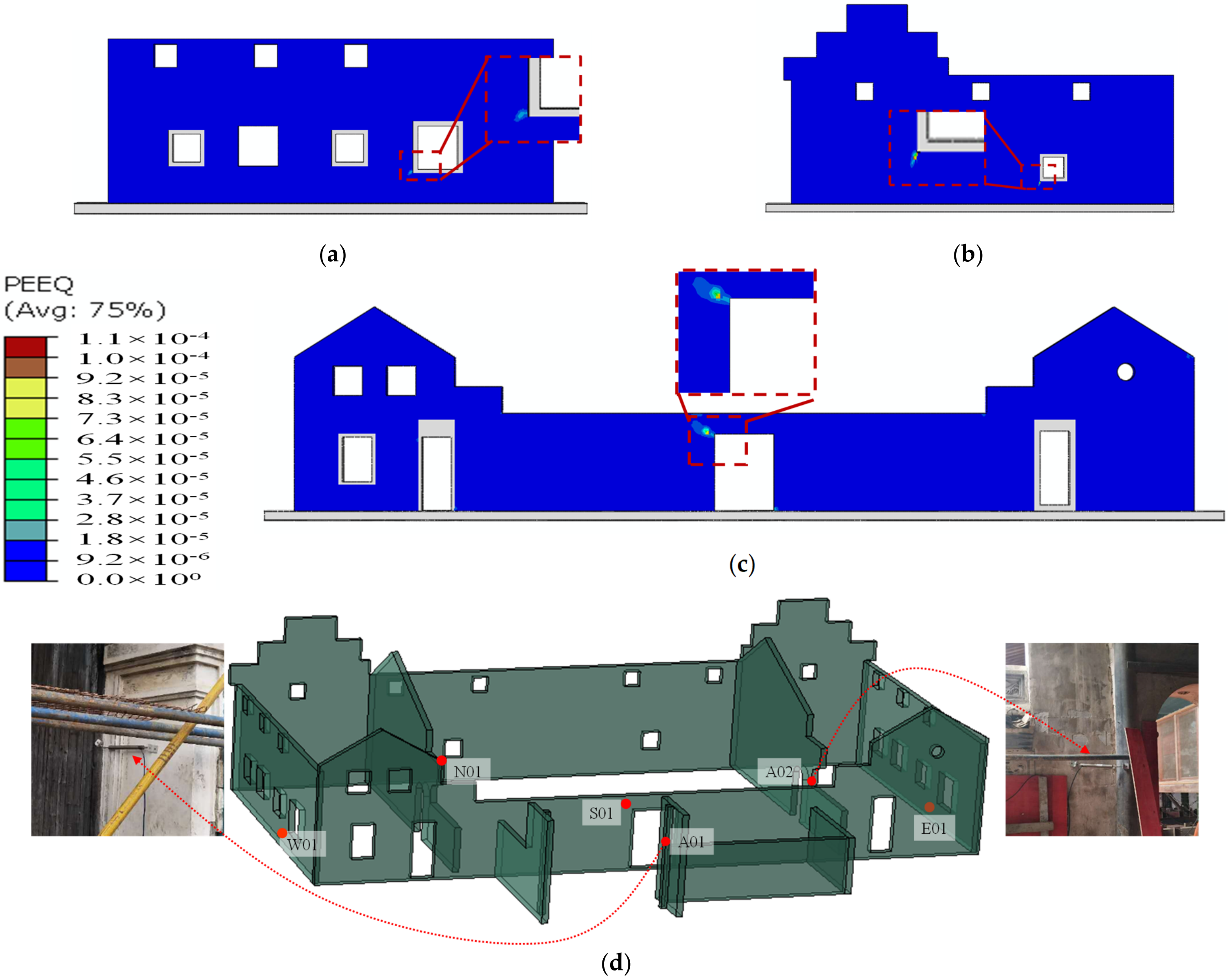


Figure 7. PEEQ distributions of the exterior walls: (**a**) west (**b**) north and (**c**) south sides; (**d**) layout of the displacement transducers.

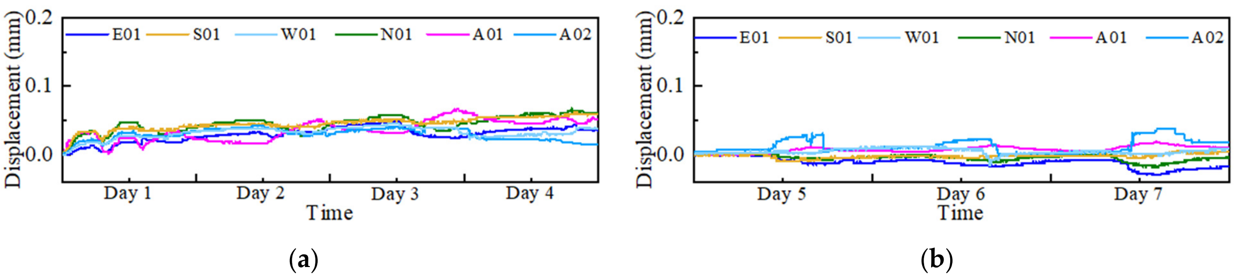


Figure 8. Crack displacements obtained from the displacement transducers: (**a**) westward; (**b**) northward.

3.3. Deformation of the Steel Underpinning Beams

In order to transfer the vertical loads to the steel chassis, the reinforced underpinnings are constructed at the bottom of the masonry walls with two-clip steel beams. As shown in Figure 8a, three serious inclinometers are stuck on the outside surfaces of the steel beams at the bottom of the east, north, and west walls in this case, assigned as D01, D02, and D03, respectively. Here, D01 and D02 are selected as representative sensors for simplicity, and the ranges of their measured lengths are 12 m along the east–west direction and 30 m along the south–north direction, respectively. The used series inclinometer with ten measuring points could output displacement data along the Y and Z directions, and their local coordinates were plotted in Figure 9a. Figure 9b presents the displacements of D01 along the Y_1 directions. Whether moving westward or northward during these 7 days, the displacement curves almost followed the same trend with the range from -0.5 to 1.2 mm, which indicates that there is almost no deformation along the Y_1 direction under the external horizontal loads provided by the push-in jacks (Figure S1). This is because the moving speed is very low and the stiffness of the underpinning beams in the Y_1 direction is high enough to resist the deformation. When it comes to the deformation along Z_1 direction in Figure 9c, the variation trends of the displacement curves are also similar; meanwhile, the displacement in the Z_1 direction decreased from 6.0 to 1.5 mm with the increased distance from the north end to the south end of the series inclinometer when moving west in the first to the fourth days. The displacement variations mainly originate from the difference of the external pushing loads. Additionally, it is obvious that the amplitude of deformation is larger than that during the movement north in the fifth to seventh days and the D01 data, which is caused by the direction of the pushing loads and the lower stiffness of the underpinning beam in the Z_1 direction than that in the Y_1 direction. Similarly, the displacement curves obtained via the D02 sensor are quite stable (small gaps of 1.2 to 2.0 mm and 0.3 to 1.2 mm for the westward and northward movements) in Figure 9d, which proves the homogeneous deformation in Y_2 direction of the underpinning beam during the whole monolithic movement. Additionally, the stable displacement curves in Z_2 direction distracted from the D02 sensor with a small variation of -0.1 to 0.9 mm can be found when moving west in Figure 9e. The larger deformation with the displacement varying from -5 to -2 mm is revealed when moving north due to a combination of the significantly lower stiffness in the Z_2 direction than the Y_2 direction and the external pushing-load direction.

3.4. Strain of the Steel Chassis

The strain variations in the steel chassis, which bears the vertical loads of the whole mobile body, are collected by the strain gauges and automatically accessed by the computer in this case. Taking one strain gauge on each side of the chassis for the typical examples, their locations are presented in Figure S1, and the corresponding strain variations are shown in Figure 9, where SE, SW, SN, and SS denote the strain gauge attached on the east, west, north, and south steel chassis beams. During both the westward and northward movements, a good correlation in the strain variations can be found for the SE and SW curves, and also for the SN and SS curves, which indicates the synchronous moving process of the opposite steel beams and further provides a guarantee for maintaining the integrity and availability of the curtilage. The strains of the north and south beams are higher than those of the east and west beams in Figure 10a, which are induced by the horizontal pushing force provided by the push-in jack during the first westward monolithic movement stage. Moreover, the strains of the east and west beams are nearly zero, which proves that there is little or no deformation of them. Similarly, the higher strain of the north and south beams and the near-zero strain of the east and west beams are shown in Figure 10b during the northward stage. Furthermore, it is interesting to notice that, despite the reasonable amplitude of these curves, the strains for all the studied chassis beams are less than 0.04%, which is far less than the yield strain of the steel material, helping to confirm that steel chassis are keeping in the elastic range during moving to the new location.

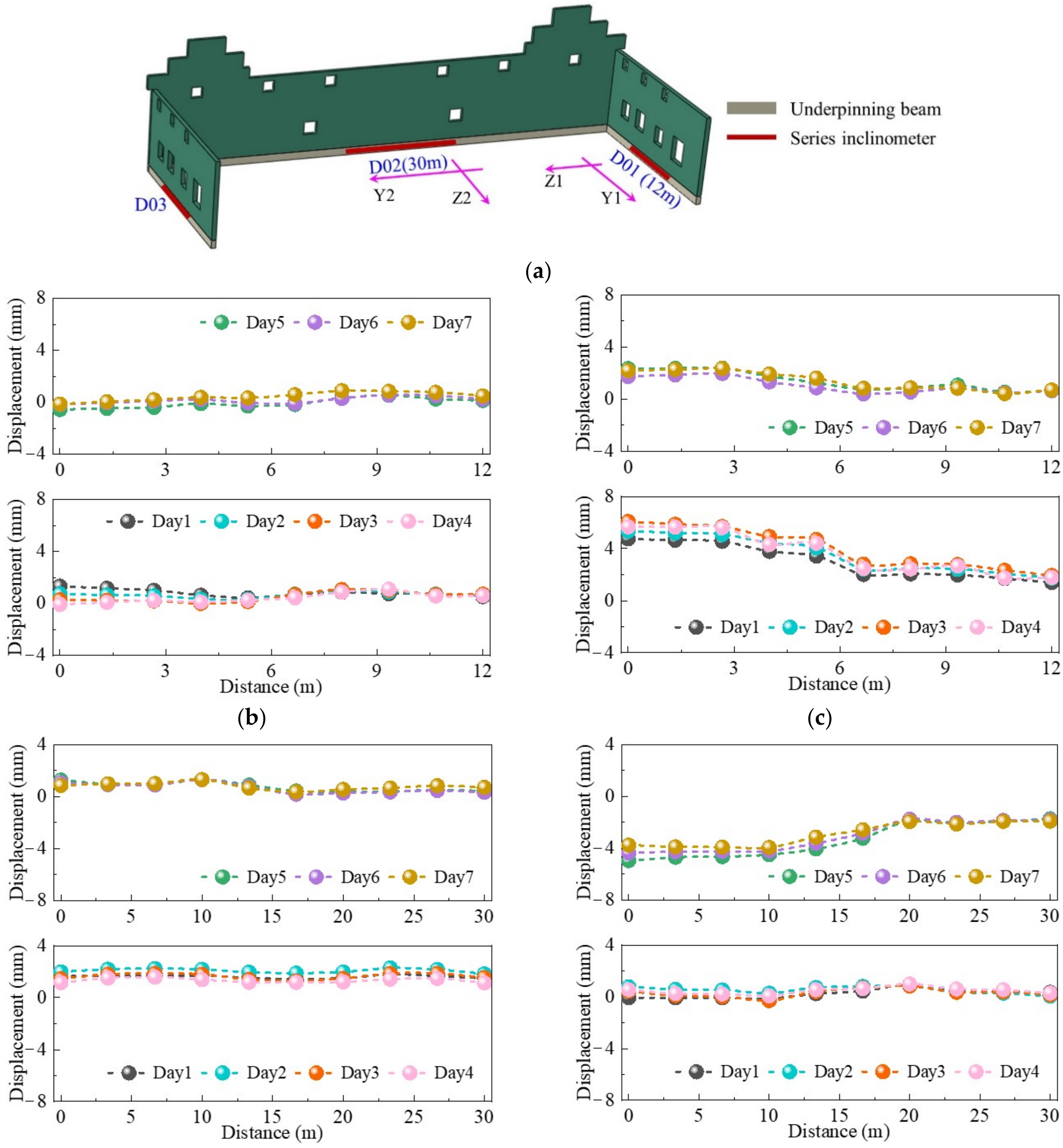


Figure 9. (**a**) The distribution of the serious inclinometers; the displacements measured by: D01 along the (**b**)Y_1 and (**c**) Z_1 directions and D02 along the (**d**) Y_2 and (**e**) Z_2 directions.

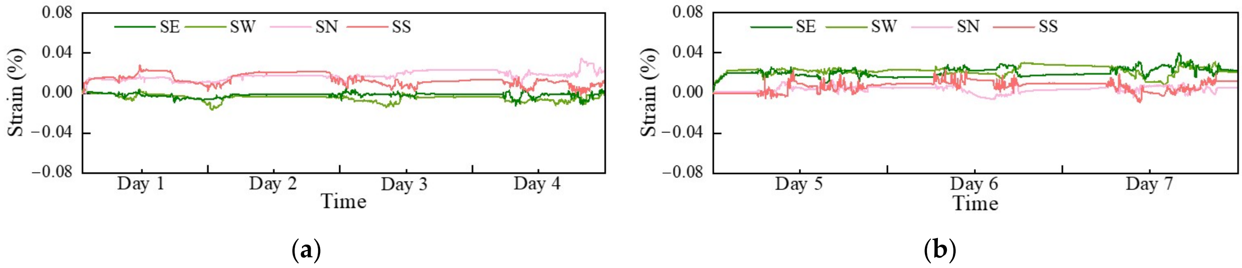


Figure 10. Strain variations during the (**a**) westward and (**b**) northward movement.

4. Conclusions

The study undertaken herein involves the case study of a WSN-based approach enabling real-time monitoring for the monolithic movement of a heritage curtilage. In consideration of the lifting jack technology to implement the monolithic movement, the deformation of the steel underpinning beams and the steel chassis are monitored simultaneously in addition to the inclination and the crack behaviors during translocation.

The following conclusions are drawn:

1. The inclinometer results show that the inclination of the timber pillars and the exterior masonry walls is almost negligible, confirming that the used lifting-jack technology has great potential in the monolithic movement of the historic building;
2. FE simulation and engineering experience are combined to reveal that the corners of the window and the door as well as the masonry-timber connections tend to crack more easily, thereby determining the distributions of the displacement transducers. The corresponding displacement curves reveal that the crack behaviors are affected by the moving direction. The value range is also changed from −0.02 to 0.07 mm, which is not enough to initiate structural cracks;
3. The strain gauges are attached to the steel underpinning beams to monitor their deformations. It is obvious that the deformation of the steel underpinning beam is related to the moving direction and its stiffness in different directions;
4. The strain variations of the steel chassis, obtained from series inclinometers, are less than 0.04%, which provides evidence of keeping the elastic range during monolithic movement.

In brief, this work provides useful insights into developing non-invasive, real-time, and remote monitoring strategies for historic buildings.

Supplementary Materials: The following supporting information can be downloaded at: https://www.mdpi.com/article/10.3390/buildings12111785/s1, Figure S1: The plane view of the steel chassis with 9 longitudinal and 16 transverse H-shaped steel beams, floating jacks, pushing loads, and typical strain gauge locations; Table S1: The input parameters of the exterior walls.

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