JCE
JOURNAL OF CIVIL ENGINEERING
ISSN 2086 - 1206 E-ISSN 2579 - 9029

# Evaluation of Damage and Strengthening Recommendations: A Case Study of the Lontar Badminton Sports Hall

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Submitted : 05 June 2026
Revised : 13 June 2026
Accepted : 20 June 2026

## Abstract

An evaluation and recommendations for strengthening the structure of the Lontar Badminton Sports Hall were conducted to assess its adequacy after damage occurred to the strauss piles and the collapse of the retaining wall. Evaluation methods included an initial survey, destructive and non-destructive testing of structural elements, and structural analysis. Material test results indicated that the compressive strength of the concrete in the upper structure averaged 22.55 MPa. In contrast, the strauss pile structure had an average compressive strength of only 6.81 MPa. This disparity suggests that the concrete quality in the substructure elements is substandard. Analysis of the steel element capacity revealed a maximum cross-sectional capacity ratio of 10.7 for the HCO and CNP profiles. This indicates that certain elements may not be able to support the design load. Additionally, five types of pile caps—PC1, PC1-A, PC2, PC4, and PC5—have been identified as unsafe due to a combination of axial loads and moments that exceed their capacity. Furthermore, the reinforced concrete floor slabs are unable to bear the existing loads because of their excessive spanning area. Recommendations for strengthening the upper structure include adding steel elements, specifically WF 200x100 longitudinal beams, and adjusting the profiles to enhance overall structural stiffness and capacity. For the lower structure, proposed modifications involve adjusting the number and dimensions of foundations and pile caps to improve the lower structural configuration.

**Keywords**

Structural evaluation, sports hall, strengthening, strauss pile, reinforced concrete structure, destructive, non-destructive testing

## INTRODUCTION

Building structures are fundamental elements in supporting the continuity of various human activities, ranging from housing and public services to commercial activities. The reliability and safety of a structure are key prerequisites for ensuring the safety of its users. Building structures must be capable of safely withstanding various loads and possess adequate safety factors (PBG Building Safety Standards) [1]. International standards, such as those issued by the International Organization for Standardization (ISO) and the European Committee for Standardization (CEN) through the Eurocodes, comprehensively regulate the reliability and safety requirements for building structures [2], [3].

In practice, events such as uneven ground settlement, soil pressure due to poor drainage, or inadequate material quality can cause fatal damage [4]. This damage often progresses over time. This situation occurred in the Lontar Badminton Sports Hall Construction Project. Based on the results of an initial survey of the existing building's condition, structural damage was identified, including damage to the strauss piles and the collapse of the retaining wall, as shown in the shop drawings in Figure 2, specifically at As 1/F-I. Similar case studies regarding pile foundation and retaining wall failures have been widely reported in the civil engineering literature, such as the analysis of pile foundation failure in multi-storey buildings in Turkey [5] and the evaluation of retaining wall collapse due to excessive lateral pressure in South Sumatra, Indonesia [6]. Therefore, a comprehensive understanding of the actual overall condition of the existing building is required.

One approach that can be taken to address the above issues is through a structural assessment [7], [8]. A structural assessment of the Lontar Badminton Sports Hall was carried out to determine the structural integrity and safety of the building following the failure of the strauss pile foundations and the collapse of the side retaining walls. The results of the structural assessment will form the basis for structural repairs or reinforcement.

The sports hall is a steel-framed structure. At the time of the damage, the building was still under construction, having reached the stage of rafter installation. The situation was exacerbated by the condition of the strauss pile foundations and the side building's pedestal columns,

ISSN 2086 - 1206 E-ISSN 2579 - 9029

specifically at axes 1/G and 1/H, which were already collapsed at the time of the initial survey. In building construction, deep foundations are a critical element in bearing design loads, as they serve to transfer loads from the upper building structure to the ground [9]. To analyse structures accurately, the use of software such as SAP2000 has become standard practice in structural engineering for modelling and calculating structural responses to various load combinations [10], [11] as demonstrated in various building structural evaluation studies [12], [13].

Based on this background, the objective of this study is to evaluate the quality of the existing Lontar Badminton Sports Hall, to re-analyse the condition of the existing structure against planned loads in accordance with applicable regulations using the SAP2000 software, and subsequently to provide recommendations for structural repairs or reinforcement.

## RESEARCH SIGNIFICANCE

This study has two main objectives, first to investigate the current condition of the building after the damage occurred; and second to recommend strengthening or repair measures to maintain the existing structure, thereby ensuring occupant safety and comfort under both normal service conditions and seismic events.

## METHODOLOGY

### A. LOCATION

The badminton sports hall building that serves as the case study in this research is located at Jalan Raya Lontar Kulon 74, Surabaya, East Java. The research location is shown in Figure 1.


Figure 1 Building location

### B. RESEARCH STAGES

This study was conducted systematically to determine the reliability of the building's existing condition. The detailed evaluation process is as follows:

1. Initial visual survey of the building structure by observing the technical condition of each structural element, including abutments, floor slabs, beams, and columns, as well as other supporting structural elements.
2. Measurement of structural dimensions, including the collection of documentation in the form of design drawings.
3. Collection of building material data through field testing, employing both destructive and non-destructive testing methods. Destructive testing involved core drilling to determine the compressive strength of reinforced concrete and tensile strength testing of reinforcing steel, whilst non-destructive testing utilised a hammer test to assess concrete uniformity.
4. Structural analysis using the SAP2000 software based on design loads and the results of tests on the building's existing materials.
5. Providing technical recommendations regarding the existing condition of the building.

## RESULTS AND DISCUSSIONS

### A. INITIAL VISUAL SURVEY

An initial visual survey was conducted to assess the existing condition of the building and identify initial damage. Based on observations of the Badminton Sports Hall, which consists of a single storey, the upper structural elements, which are still under construction, are in good condition, whilst the lower structural elements exhibit structural failure in the retaining walls and strauss pile foundations. The strauss pile foundations that had been constructed had failed, breaking (Figure 3), and the retaining walls had slipped (Figure 4). Referring to the design drawings (see Figure 2), the failure occurred at axis 1/F-I.

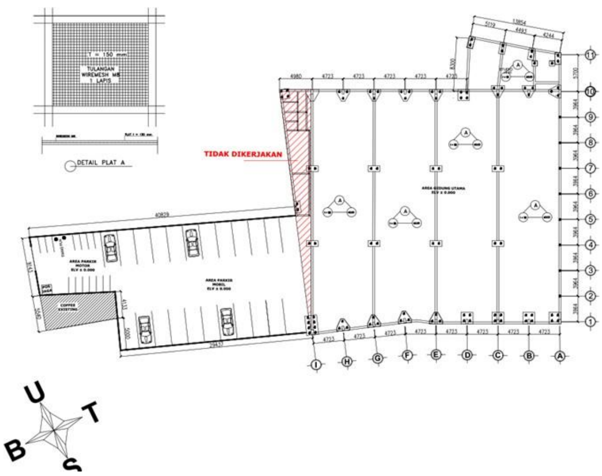



Figure 2 Foundation design plan

Figure 3 Identification of strauss pile failure

Figure 4 Identification of retaining wall failure

B. STRUCTURAL ELEMENT TESTING

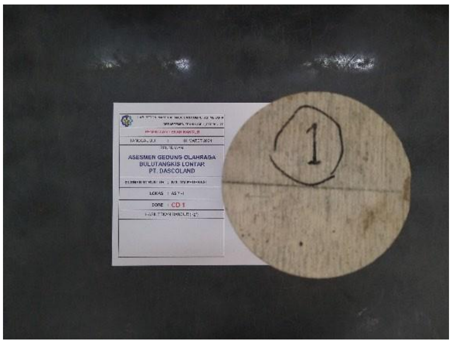
Structural element testing was carried out to determine the strength of the materials in their existing condition. The testing was divided into two categories: destructive and non-destructive. Destructive testing involved core drilling of the concrete, whilst non-destructive testing comprised hammer tests and tensile testing of the steel reinforcement.

CORE DRILL

This test is carried out to determine the compressive strength of the concrete core drill cylinder, thereby establishing the quality of the concrete currently in place. The testing is carried out in accordance with ASTM C42/C42M [14]. A total of 6 (six) core drill concrete cylinder test specimens were taken. The test specimens were taken from the structural elements of the pedestal columns, sloof and strauss piles of the badminton sports hall, and were subsequently taken to the laboratory for compressive strength testing.

Prior to conducting the compressive strength test, the concrete core drill samples are measured and their surface condition examined. The measurements and surface condition of the concrete core drill samples are recorded on the core log sheet. From these records, the number and position of the reinforcing bars remaining within the concrete core drill samples are identified; this information is subsequently used to determine the conversion of the concrete sample's compressive strength due to the remaining reinforcing bars. Figure 5 shows an example of a test specimen and the process of conducting the compressive strength test on a concrete core drill cylinder sample at point 1 on the pedestal column.


(a)




(b)

Figure 5 Core drill test specimen CD-1: (a) side view and (b) top view

The results of the core drill test can be seen in Table 1 and Table 2. The average compressive strength of concrete for the upper structure is 22.55 MPa. In contrast, the average compressive strength for the lower structure, specifically the strauss piles, is 6.81 MPa.

Table 1 Core drill test results

| Code | CD-1 | CD-2 | CD-3 | CD-4 |
|---|---|---|---|---|
| Element | Pedestal column | Pedestal column | Pedestal column | Sloof |
| Axis Loc. | 7/I | 7/H | 7/G | 7-8/C |
| Compressive strength of cylinders Ø15–30 cm (MPa) | 22.69 | 25.36 | 24.83 | 17.30 |
| Average (MPa) | 22.55 | | | |

Table 2 Core drill test results (continue)

| Code | CD-5 | CD-6 |
|---|---|---|
| Element | Strauss pile | Strauss pile |
| Axis Loc. | 1/F | 1/G |
| Compressive strength of cylinders Ø6.8 cm (MPa) | 4.33 | 9.29 |
| Average (MPa) | 6.81 | |

HAMMER TEST

This test was conducted to predict the uniformity of concrete quality. A total of 22 field samples were taken. The tests were carried out on the concrete surfaces of the pedestal columns, sloof and strauss piles of the badminton sports hall. Table 3 below shows the rebound hammer test results. The results obtained from the core drill and rebound hammer tests conducted at points CD-1 through CD-4 were utilised to estimate the compressive strength of the concrete at each test location. A linear regression analysis was performed on the test data (see Figure 6), resulting in a predicted average compressive strength of 246.83 kg/cm² for the upper structure, which includes the pedestal columns and sloof, and 163.56 kg/cm² for the lower structure, comprising the straus piles.

Table 3 Hammer test results

| No | Structural Element | Axis Location | Angle | Average Rebound | Compressive strength prediction (MPa) |
|---|---|---|---|---|---|
| 1 | Pedestal column | 7/I (CD-1) | 0 | 36.00 | 22.80 |
| 2 | Pedestal Column | 7/H (CD-2) | 0 | 39.40 | 27.62 |
| 3 | Pedestal Column | 7/G (CD-3) | 0 | 38.00 | 25.63 |
| 4 | Pedestal column | 7/F | 0 | 36.80 | 23.93 |
| 5 | Pedestal Column | 7/E | 0 | 36.00 | 22.80 |
| 6 | Pedestal Column | 7/D | 0 | 38.20 | 25.92 |
| 7 | Pedestal column | 7/C | 0 | 36.40 | 23.37 |
| 8 | Pedestal Column | 7/B | 0 | 36.00 | 22.80 |
| 9 | Pedestal Column | 7/A | 0 | 37.40 | 24.78 |
| 10 | Pedestal column | 8/D | 0 | 36.80 | 23.93 |
| 11 | Pedestal Column | 8/C | 0 | 36.40 | 23.37 |
| 12 | Pedestal Column | 8/B | 0 | 36.00 | 22.80 |
| 13 | Pedestal column | 8/A | 0 | 39.60 | 27.90 |
| 14 | Pedestal Column | 1/E | 0 | 37.00 | 24.22 |
| 15 | Pedestal Column | 1/D | 0 | 37.60 | 25.07 |
| 16 | Pedestal column | 1/C | 0 | 36.80 | 23.93 |
| 17 | Pedestal Column | 1/B | 0 | 36.00 | 22.80 |
| 18 | Pedestal Column | 1/A | 0 | 36.40 | 23.37 |
| 19 | Sloof | 7-8/C (CD-4) | 0 | 36.80 | 23.93 |
| 20 | Sloof | 7-8/D | 0 | 36.20 | 23.08 |
| 21 | Strauss pile | 1/G (CD-5) | 0 | 25.60 | 16.84 |
| 22 | Strauss pile | 1/F (CD-6) | 0 | 24.40 | 15.24 |

Table 3 shows the results of the rebound hammer test at 22 points, while the predicted compressive strength values were obtained from the regression equation shown in Figure 6. The predicted compressive strength values based on the hammer test results for pedestal columns and sloof range from 22.80 MPa to 27.90 MPa. The average predicted compressive strength is 24.20 MPa. Meanwhile, the results for strauss pile rebound readings ranged from 24.40 to 25.60, whilst the average rebound reading was 25.00. The predicted compressive strength values based on the hammer test results ranged from 16.84 MPa to 15.24 MPa. Thus, the average predicted compressive stress value for the lower structural elements was 16.04 MPa.

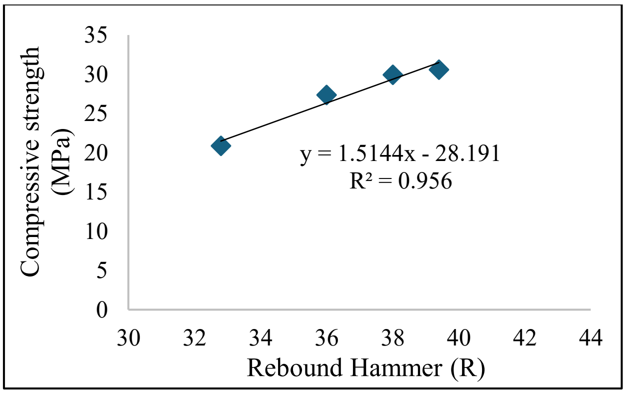



Figure 6 Regression analysis of the relationship between core drill compressive strength and hammer test

TENSILE TEST OF REINFORCING STEEL

This test was conducted to determine the yield strength ($f_y$), tensile strength ($f_u$ ) and ultimate strain of the reinforcing steel. A total of 8 (eight) samples were taken for this test. Based on the results of the tensile strength test of the reinforcing steel, it was found that the plain concrete bars Ø8 mm and Ø10 mm fall into the BJTP 280 classification and the D16 threaded concrete bars into the BJTS 420 classification. Meanwhile, the D13 threaded concrete bars fall into the BJTS 280 classification. The test results for the tensile strength of reinforcing steel are shown in Figure 7 and Table 4.

Figure 7 Reinforcement tensile test specimen

Table 4 Test results for tensile strength of reinforcing bars

| No | Weight (kg/m') | Diameter (mm) | Area ($mm^2$) | Yield strength, $f_y$ (MPa) | Tensile strength, $f_s$ (MPa) | Fracture strain (%) |
|---|---|---|---|---|---|---|
| 1 | 0.348 | 7.51 | 44.30 | 421.78 | 676.80 | 22.11 |
| 2 | 0.348 | 7.51 | 44.30 | 433.92 | 675.28 | 23.61 |
| 3 | 0.573 | 9.64 | 72.98 | 365.66 | 567.44 | 24.23 |
| 4 | 0.564 | 9.57 | 71.93 | 371.94 | 577.61 | 26.20 |
| 5 | 0.944 | 12.38 | 120.37 | 358.00 | 521.12 | 21.50 |
| 6 | 0.943 | 12.37 | 120.21 | 362.40 | 563.21 | 27.70 |
| 7 | 1.504 | 15.62 | 191.75 | 488.80 | 664.85 | 24.49 |
| 8 | 1.517 | 15.69 | 193.43 | 481.78 | 661.85 | 22.82 |

## C. STRUCTURAL ANALYSIS

The structural analysis of the Lontar Badminton Sports Hall was carried out using the finite element method with the SAP2000 software. The analysis was conducted as a review of the dimensions/sizes of the profiles already installed on-site, with loads in accordance with the building's design. The loads used in the analysis include dead loads, live loads, wind loads, and rainwater loads, whilst the load combinations refer to SNI 1727-2020 [15]. The building geometry refers to the design drawings as shown in Figure 2, whilst the material quality refers to the results of the on-site investigation.

### Structural Material Properties

The material properties used in the structural analysis process are obtained from field testing, as described in Subsection B, and can be seen in Table 5.

Table 5 Structural element materials

| Element | Material Quality |
|---|---|
| Pedestal column | $f'_c$ = 18.67 MPa |
| Pile cap; strauss pile | $f'_c$ = 7 Mpa |
| Steel reinforcement for D16 and larger sizes | $f_y$ = 420 MPa |
| Steel reinforcement for D13, Ø10, Ø8 and sizes below | $f_y$ = 280 MPa |
| Steel | BJ 37 $f_y$ = 240 MPa; $f_u$ = 370 MPa |

LOADING

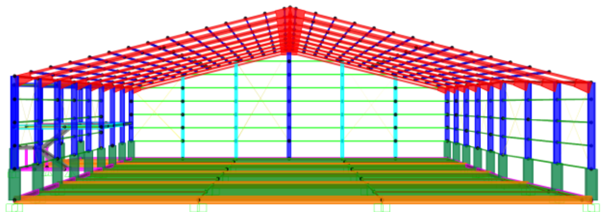
The loads on the building, based on design data in accordance with SNI 1727-2020 [15], are shown in Table 6.

Table 6 Structural loads

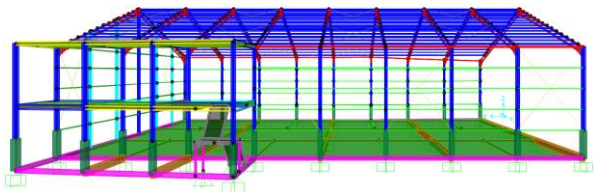
| Loads | Value |
|---|---|
| Self-weight of steel | 7850 kg/$m^3$ |
| Self-weight of concrete | 2400 kg/$m^3$ |
| Additional dead load of the roof | 11 kg/$m^2$ |
| Wind load | 25 kg/$m^2$ |
| Rainwater load | 20 kg/$m^2$ |
| Live load for badminton court | 100 kg/$m^2$ |
| Live load on studio floor | 488.5 kg/$m^2$ |
| Live load on stairs | 400 kg/$m^2$ |

### Structural Modelling

Structural analysis in the SAP2000 software assumes that the structural system is a *space frame* model (3D *frame system*). The geometric model of the building's main structure is shown in Figure 8.



(a)



(b)

Figure 8 Geometric modelling of the building using SAP2000: (a) Front view and (b) side view

## D. STRUCTURAL ANALYSIS RESULTS

The structural analysis results presented consist of two elements, namely steel structural elements and reinforced concrete structural elements. The results of the analysis of steel structural elements presented are the results of modelling using SAP2000, whilst the results for reinforced concrete structural elements are the results of internal force analysis from SAP2000 and capacity analysis using Ms Excel and SPColumn.

### Steel Element Capacity Ratio Analysis

The analysis of the strength of the steel profiles used is based on LRFD (*Load and Resistance Factor Design*). This method is based on the concept of *limit states*, namely a condition in which a structure or structural element is designed to the point where it can no longer function. There are two categories that define *limit states*:

1. Strength limit state: the structure's ability to bear loads
2. Serviceability limit state: the behaviour of the structure under load

The ratio shown is the comparison between performance and capacity, whereby the requirement is that the capacity must be greater than the structural performance, resulting in a ratio of less than 1 (one). The results of the ratio analysis can be seen in Figure 9. Most elements are safe, as indicated by a ratio of less than 1 (one); however, there are several elements in the bracing, beams and columns that are overstressed, indicating an inability to withstand the design load. The HCO and CNP elements are the most critical with ratios of up to 10.7, due to high lateral and axial loads, whilst the WF columns are overstressed due to P-M interaction.

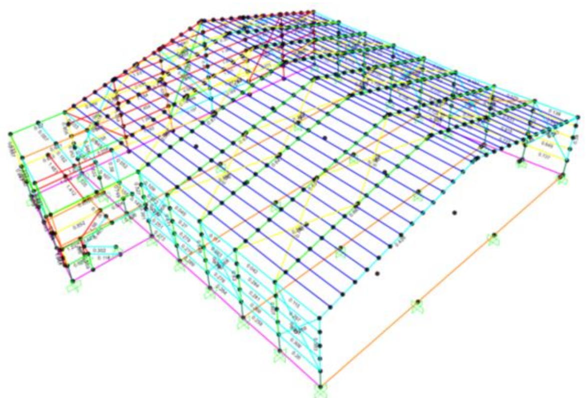


Figure 9 Results of the steel capacity ratio analysis

### Floor Slab Analysis

Based on the results of structural modelling and analysis using the SAP2000 programme for the existing conditions, it was found that the floor slab capacity of the Lontar Badminton Sports Hall does not meet technical safety requirements. This is due to the placement of the footings, which does not optimally support the slab area.

Technically, the floor slab has a span that is too wide without any intermediate supports. Consequently, the bending moment resulting from the design load ($M_u$) far exceeds the nominal moment capacity ($M_n$) of the currently installed reinforcement. As a result, the floor slab structure is deemed unsafe. A summary of the floor slab calculation results can be seen in Table 7.

Table 7 Summary of floor slab calculations

| Parameter | Value |
|---|---|
| Dimensions | 37330 x 9446 x 150 mm |
| Design load in X-direction, $M_{ux}$ | 8,955 kNm |
| Design load in Y-Direction, $M_{uy}$ | 68,883 kNm |
| Reinforcement Installed in X- direction | M8 – 150 |
| Installed reinforcement in the Y-direction | M8 – 150 |
| Required reinforcement in the X-direction | M8 – 150 |
| Reinforcement Required in Y- direction | M8 – 30 |
| Status | Failed/Unsafe |

## E. SLOOF BEAM ANALYSIS

Structurally, the sloof beam serves to connect the foundations and distribute the wall loads. However, the analysis results indicate that for the S1 (200x300), the bending moment ($M_u$) and shear force ($V_u$) exceed the capacity of the existing reinforced concrete cross-section, meaning the sloof beam structure is deemed unsafe. A summary of the sloof beam calculation results can be seen in Table 8 and Table 9.

Table 8 Summary of sloof beam calculations

| Types | Dim. (mm) | Support bar | | Mid-span bar | |
|---|---|---|---|---|---|
| | | Top | Bot. | Top | Bot. |
| S1 | 200x300 | 2D16 | 2D16 | 2D16 | 2D16 |
| S2 | 200x300 | 2D16 | 2D16 | 2D16 | 2D16 |
| S3 | 300x500 | 4D16 | 4D16 | 4D16 | 4D16 |

Table 9 Summary of sloof beam calculations (continue)

| Types | Dim. (mm) | Shear bar | Status |
|---|---|---|---|
| S1 | 200x300 | Ø10-100 | Unsafe |
| S2 | 200x300 | Ø10-100 | Safe |
| S3 | 300x500 | Ø10-100 | Safe |

### Analysis Of Pile Caps and Strauss Pile

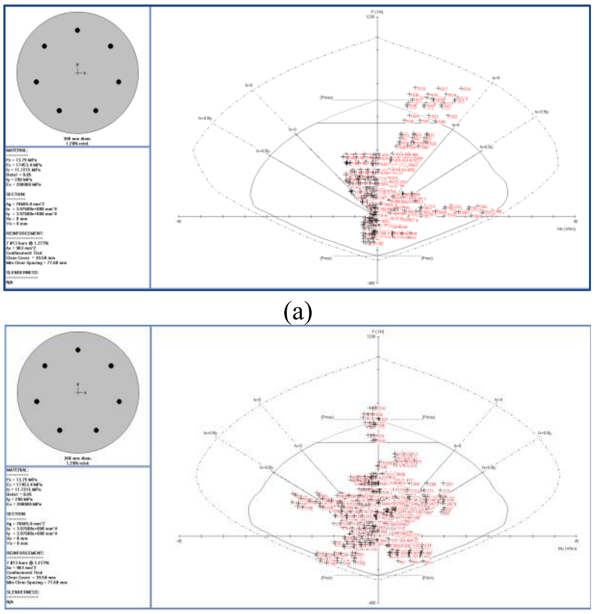
The analysis of the substructure includes an evaluation of the pile caps and strauss piles. Based on the results of material testing on-site and load analysis using SAP2000, it was found that the substructure had failed.

The primary cause of this failure is the low compressive strength of the existing concrete, which does not meet technical standard specifications. This poor concrete quality resulted in the strauss pile foundations being unable to bear the axial load and moment from the superstructure, as evidenced by the physical finding of a fractured foundation as shown in Figure 3 and Figure 4. Furthermore, the configuration of the existing pile cap was unable to provide sufficient bearing capacity for the applied loads, causing the stresses to exceed the allowable soil capacity. A summary of the calculation results for the lower structure's capacity can be seen in Table 10.

Table 10 Summary of pile cap calculations

| Type | Number of Piles | Maximum Capacity Ratio of Piles per Type | Status |
|---|---|---|---|
| PC 1 | 3 | 1.42 | Unsafe |
| PC 1-A | 3 | 1.03 | Unsafe |
| PC-2 | 2 | 1.92 | Unsafe |
| PC-3 | 1 | 0.77 | Safe |
| PC-4 | 2 | 1.64 | Unsafe |
| PC-5 | 4 | 1.39 | Unsafe |

Analysis of the strauss pile capacity using the SPColumn software indicates that the reinforcement installed in the strauss piles is unable to withstand the applied loads. This is evident from the numerous points lying outside the line of interaction between axial force and moment. The analysis results are shown in Figure 10.



(a)

(b)

Figure 10 Cross-section analysis of strauss pile: (a) X-direction and (b) Y-direction

**Summary Of Structural Calculations**

The results of the structural calculation summary, as described previously, are summarised in Table 11.

## F. STRUCTURAL REPAIR RECOMMENDATIONS

Based on the results of the structural analysis identifying elements deemed unsafe (capacity ratio >1) in the Lontar Badminton Sports Hall structure, the strengthening recommendations focus on enhancing the capacity of critical elements through an efficient and economical retrofitting approach.

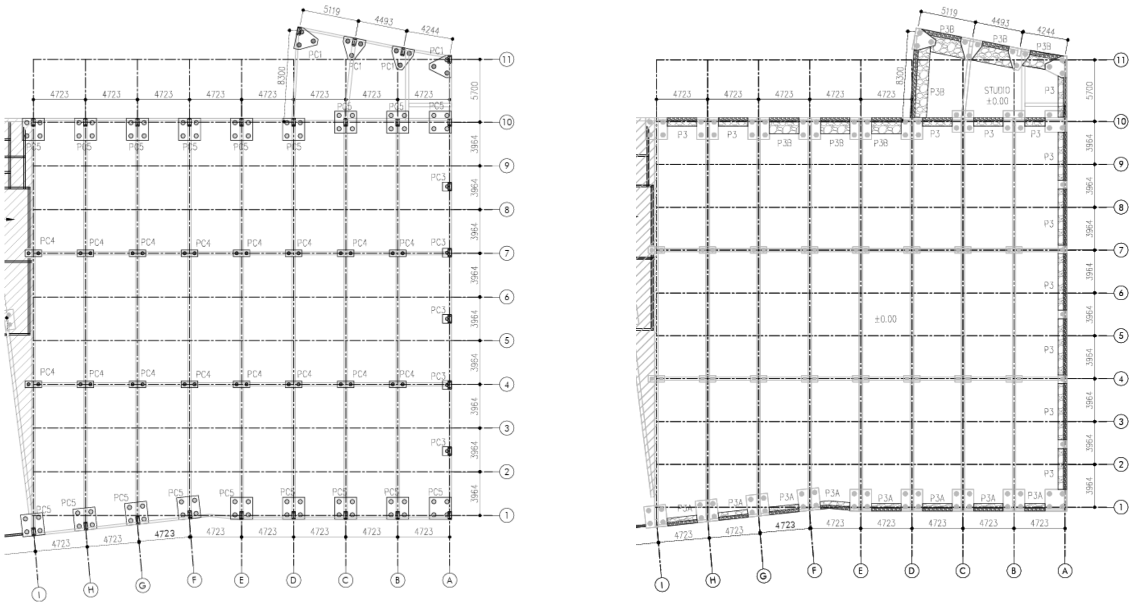
To achieve a safe structural configuration that meets technical standards, design modifications have been made to the entire structural system. These changes include the re-determination of the number and dimensions of the lower structure, as well as adjustments to the configuration of the upper structure. Specifically, improvements to the lower structure focus on strengthening the footing and foundation elements to ensure they meet the available soil bearing capacity.

Table 11 Summary of structural analysis

| Structural Element | Result | Remarks |
|---|---|---|
| HCO 525x175x7x11 | ☒ | Capacity ratio >1 |
| Castella 525x175x7x11 | ☑ | - |
| Castella 450x150x6.5x9 | ☑ | - |
| Castella 375x125x6x9 | ☑ | - |
| Castella 300x100x5.5x8 | ☑ | - |
| Castella 225x75x5x7 | ☒ | Capacity ratio >1 |
| WF 450 x 200 x 9 x 14 | ☒ | Capacity ratio >1 |
| WF 350x175x7x11 | ☒ | Capacity ratio >1 |
| WF 200x100x5.5x8 | ☑ | - |
| CNP 100x50x20x2.3 | ☒ | Capacity ratio >1 |
| CNP 125x5 | ☑ | - |
| Double CNP 150x50x20x2.3 | ☑ | - |
| UNP 200.80.7.5.11 | ☑ | - |
| Wind Binding D16 | ☒ | Capacity ratio >1 |
| Floor Slab | ☒ | M8 reinforcement is unable to withstand the design load |
| 20/30 Sloof Beam | ☒ | 2D16 reinforcement cannot withstand the design load |
| 30/50 Sloof Beam | ☑ | - |
| Pile Cap 1 | ☒ | Capacity ratio >1 |
| Pile Cap 1 A | ☒ | Capacity ratio >1 |
| Pile Cap 2 | ☒ | Capacity ratio >1 |
| Pile Cap 3 | ☑ | - |
| Pile Cap 4 | ☒ | Capacity ratio >1 |
| Pile Cap 5 | ☒ | Capacity ratio >1 |
| Strauss Pile | ☒ | Strauss pile unable to bear design load |

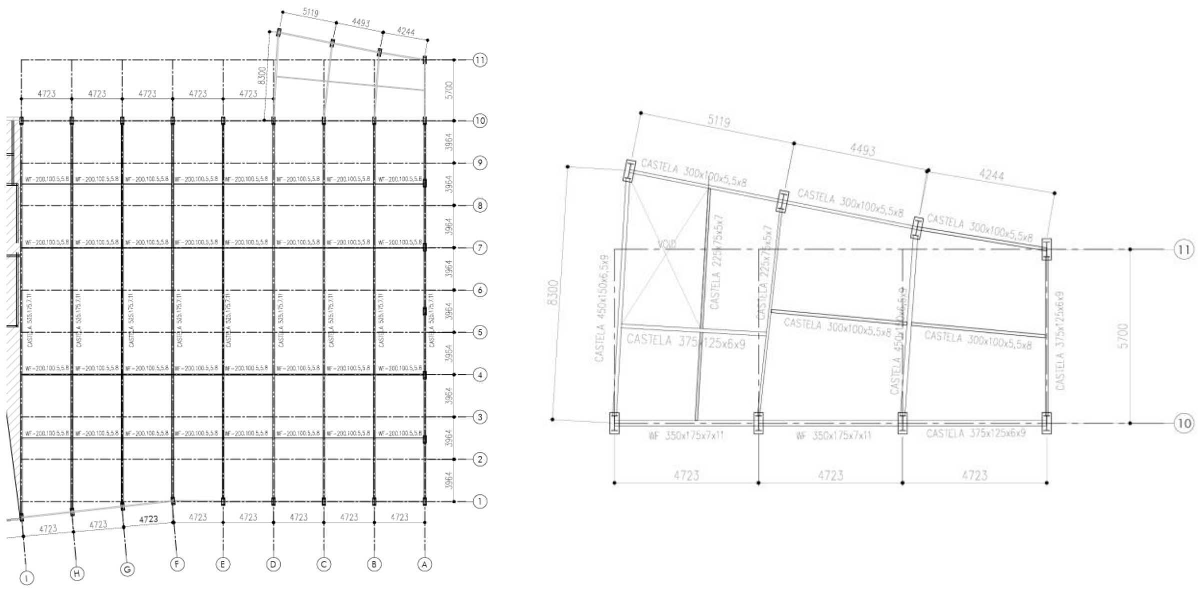
In determining the recommendations for the upper structure (steel structural elements), a structural re-analysis of the existing structure was carried out with the addition of reinforcement based on the results as shown in Figure 13. Once the recommendations for the upper structure have met the requirements, recommendations for the lower structure are then made before being incorporated into the Detailed Engineering Design (DED). Based on a re-analysis using SAP2000, the steel structural elements were found to meet the requirements overall. Subsequently, planning recommendations were made for the lower structure.

Additions and changes to the pile cap foundation configuration were made at each column point to ensure that the resulting bearing capacity could optimally support the design loads. In addition, the number of beams was increased; these serve not only to assist the foundations in distributing the load but also to reduce the area of the floor slab. With this reduction in area, the floor slab reinforcement is expected to withstand the design load more effectively. The floor slab span is divided by footings in the longitudinal direction every 11890 mm and in the transverse direction every 4723 mm. At each point where footings meet, strauss pile foundations are provided, the number of which has been updated and adjusted to the soil bearing capacity under existing conditions. Recommendations for strengthening the substructure can be seen in Figure 11.



(a) New pile cap plan (b) New sloof plan

Figure 11. Recommendations for changes to the lower structure configuration



(a) Roof truss plan for sports hall (b) Roof truss plan for studio 2nd floor

Figure 12. Recommendations for upper structure reinforcement

Meanwhile, repairs to the upper structure were carried out based on the results of an analysis using SAP2000 software. Elements with capacity ratios exceeding the permissible limit (capacity ratio >1) were reinforced by adding new steel structural elements (see Figure 12). In the rafter section, additional elements in the form of longitudinal beams with a WF 200x100x5.5x8 steel profile were installed. This reinforcement measure aims to increase the stiffness and strength of the rafters in resisting the lateral loads acting on the building, thereby maintaining the overall structural integrity.

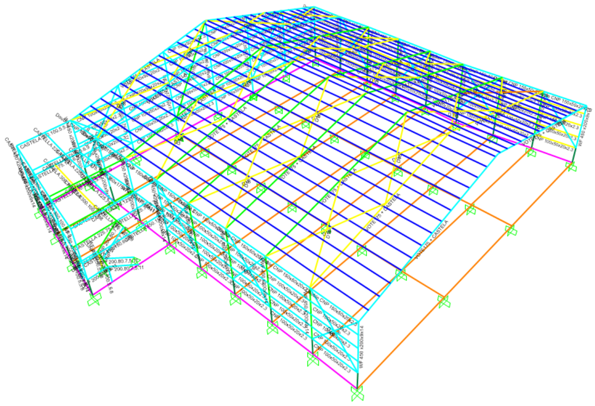


Figure 13 Results of the reanalysis of the steel capacity ratio for the recommended structure

## CONCLUSIONS

Based on the results of the structural evaluation and analysis of the Lontar Badminton Sports Hall, the following conclusions were drawn:

1. From field investigation, core drill test results showed that the compressive strength of the upper structure (pedestal columns and sloof beams) ranged from 17.30 to 25.36 MPa (average 22.55 MPa), while the substructure (strauss piles) reached only 4.33–9.29 MPa (average 6.81 MPa). Rebound hammer tests supported these findings, with predicted average compressive strengths of 24.20 MPa for the upper structure and 16.04 MPa for the substructure, indicating a significant reduction in material quality in the substructural elements.
2. Steel capacity ratio analysis show a maximum value of 10.7 for HCO 525x175x7x11 and CNP profiles. A total of six steel member types were identified as overstressed: HCO 525x175x7x11, Castella 225x75x5x7, WF 450x200x9x14, WF 350x175x7x11, CNP 100x50x20x2.3, and D16 wind bracing. These members were unable to resist the combined lateral and axial loads according to SNI 1727-2020.
3. Of the six pile cap types (PC1, PC1-A, PC2, PC3, PC4, PC5), five were unsafe with capacity ratios ranging from 1.03 to 1.92 (PC2 highest at 1.92). Only PC3 was safe (ratio 0.77). P-M interaction analysis using SPColumn showed load points lying outside the interaction curve, confirming the inadequacy of the existing strauss piles.
4. Strengthening recommendations proposed in this study, includes: (1) Modify the pile cap configuration by adding and resizing foundations, and subdivide the floor slab span into maximum panels of 11,890 mm (longitudinal) x 4,723 mm (transverse) by adding foundation beams and (2) Add longitudinal steel beams with WF 200x100x5.5x8 profiles to the rafters, and adjust other profile dimensions to enhance lateral stiffness.
5. Re-analysis using SAP2000 confirmed that all strengthened steel members achieved a capacity ratio < 1.

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