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Structural Capacity Assessment of Cremona Steel Bracing and Connections for Coastal Embankment Strengthening: A Case Study of Muara Baru, Jakarta

Kusuma Indrasurya[1], Kohar Yudoprasetyo[2], Yerry Kahaditu Firmansyah[3]
[1]Student of Civil Infrastructure Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, Indonesia
[2]Department Civil Infrastructure Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, Indonesia
[3]Department Civil Engineering, UPN Veteran Jawa Timur, Surabaya, Indonesia
[1]kindrasurya@gmail.com, [2]kohar.yudo@its.ac.id[*], [3]yerry.kahaditu.ts@upnjatim.ac.id

Abstract

Damage to the coastal protection embankment at Muara Baru required a strengthening system capable of controlling wall response while providing a structural load path that could be verified at both member and connection levels. This case study evaluates the capacity of a Cremona-type steel bracing system and its connections using data from an actual rehabilitation project. A global SAP2000 model was developed using structural loads transferred from PLAXIS 2D geotechnical analysis. Piles and tiebacks were represented by frame elements, the pilecap by shell elements, and soil-structure interaction by soil springs. Steel-member adequacy was assessed using a demand-to-capacity ratio limit of 1.0. Local connection checks were subsequently performed using finite-element analysis in IDEA StatiCa for two critical details: the WF 200×200-to-pilecap connection and the WF 200×200 Cremona bracing connection. The global model indicated that all steel members remained below the specified unity ratio. For the 1200-mm-diameter spun pile, the maximum axial utilization ratio was 0.211, whereas the maximum bending utilization ratio reached 0.933 based on the project capacity-check table. Both local steel connection details satisfied the reported acceptance criteria. The principal connection configuration comprised ASTM A572 Grade 50 plates, ASTM A325 bolts, ASTM A307 anchors, and 16-mm gusset and base plates. The results show that a staged assessment from global structural response to local connection verification provides a practical means of checking force-transfer continuity in pile-supported coastal embankment strengthening and highlights bending demand as the more critical response for the spun-pile wall.

Keyword: Cremona Bracing; Steel Connection; Coastal Embankment; SAP2000; Structural Strengthening

1. Introduction

Pile-supported coastal protection structures are subjected to combined earth pressure, variations in water level, surcharge loading, and forces that evolve during construction stages. Following damage, a strengthening scheme cannot be assessed solely from global geotechnical stability. The load path from the retaining wall to the tieback, steel bracing, pilecap, and foundation must also be verified so that each component can transfer the imposed demand without creating a new local weakness at the connections.

The need to integrate geotechnical response with structural evaluation is supported by recent studies on marine infrastructure and embankments on soft ground. Irmawan et al. (2024) used finite-element analysis to evaluate the influence of a causeway embankment on buried pipelines in soft soil and demonstrated the importance of staged embankment modelling for estimating structural demand. For pile-supported pier structures, Irwansyah et al. (2026) showed that the representation of soil-structure interaction can alter demand-capacity ratios and lateral response compared with a virtual fixed-base idealization. Darmawan et al. (2026) likewise showed that soil-structure interaction and ground conditions influence jetty performance in marine environments. These findings support the PLAXIS 2D-SAP2000 workflow adopted in this study, in which forces developed in the embankment system are traced from soil response to the strengthening members and their connections.

In braced steel structures, gusset-plate connections and bolt groups are critical components of the load path. The comprehensive review by Song et al. (2023) showed that gusset-plate capacity is governed not only by effective section area but also by block shear, boundary conditions, plate buckling, and interaction with framing members. Experimental results reported by Zhang et al. (2023) demonstrated that bolt slip and plastic connection deformation can affect bracing response. Numerical work by Xie and Zhang (2024) further showed that force distribution within gusset connections is non-uniform and strongly influenced by the brace load-transfer mechanism. More recent studies have also highlighted the influence of connection detailing on net-section fracture and gusset-plate buckling (Shin & Kim, 2025; Zheng et al., 2026).

In current design practice, complex steel connections are increasingly assessed using advanced finite element approaches to capture local stress concentrations, plate deformation, and the interaction between bolts, welds, anchors, and connected members. Morido-García and De Santos-Berbel (2024) demonstrated that detailed numerical modelling can provide a more comprehensive representation of connection behaviour than simplified analytical checks, particularly in identifying localized stress and deformation patterns. Similar numerical approaches have been widely applied to gusset plates, base plates, and bolted bracing connections to evaluate component-level response and failure mechanisms. However, published case studies that directly link the global structural response of coastal-embankment strengthening systems with detailed local verification of steel-bracing connections remain limited, particularly for rehabilitation projects in Indonesia.

For steel-to-concrete connections, Nawar et al. (2024) showed that base-plate thickness and anchor-bolt diameter and arrangement directly affect rotational stiffness and failure mechanisms. The parametric study by Khani et al. (2025) demonstrated that anchor patterns and stiffeners can alter the strength, stiffness, and force-distribution efficiency of base plates, while Jiang and Billah (2025) showed that anchor layout and bolt diameter also affect flexural capacity, ductility, and energy dissipation under multidirectional loading. These findings are directly relevant to Detail 1 in the Muara Baru case, where force transfer from the WF 200×200 member to the pilecap occurs through a base plate, anchor group, and supporting concrete.

This paper presents the strengthening of the Muara Baru coastal protection embankment as an engineering case study. The scope is intentionally limited to the structural performance of the Cremona steel bracing system and its connection details. The objectives are to: (1) describe the transfer of loads from geotechnical analysis to the structural model; (2) evaluate the capacity of the principal members using demand-to-capacity ratios; and (3) examine two critical connection details within the strengthening system. The contribution is therefore not a new numerical formulation, but an integrated case-study workflow that links geotechnical load transfer, global member verification, and local connection assessment for an actual coastal rehabilitation project.

2. Methods

2.1. Case study and strengthening configuration

The study concerns the strengthening of the Muara Baru coastal protection embankment in North Jakarta. Following damage to the embankment segment, the rehabilitation system was designed using a combination of replacement spun piles, a capping beam, landward steel pipe piles, and a triangulated/Cremona steel bracing frame acting as a tieback system. A typical strengthening section at STA 0+900 is shown in Figure 1. The drawing indicates that the steel frame provides the load-transfer path between the retaining wall and the rear foundation system.

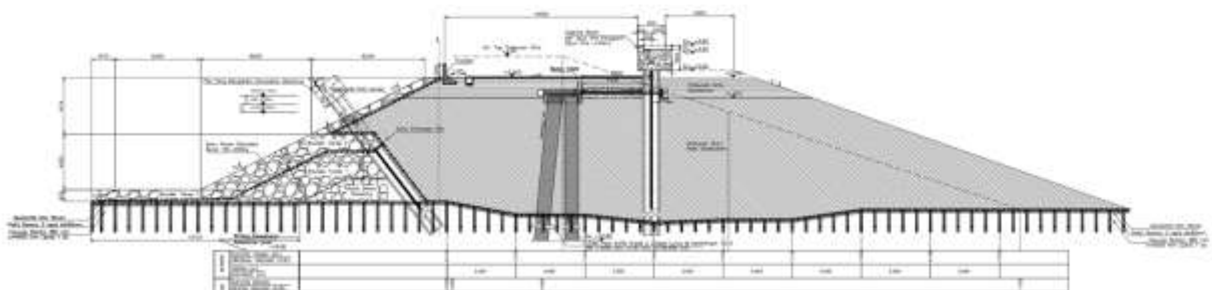

Figure 1. Typical cross-section of the strengthening system at STA 0+900

The wall region consists of a capping beam and replacement spun pile connected to the landward strengthening system. At the rear, the steel pipe pile acts as a foundation element receiving forces from the tieback-bracing system. Figure 2 clarifies the geometric relationship among the components subsequently represented in the structural model.

The Cremona frame is repeated along the longitudinal direction of the embankment. The design drawing shows a triangulated arrangement connecting the upper part of the system to the base beam supported by the pile group. In this paper, the drawing is used as the basis for interpreting the load path and the locations of critical connections,

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while numerical capacity is evaluated using the SAP2000 model and the connection checks documented in the design report.

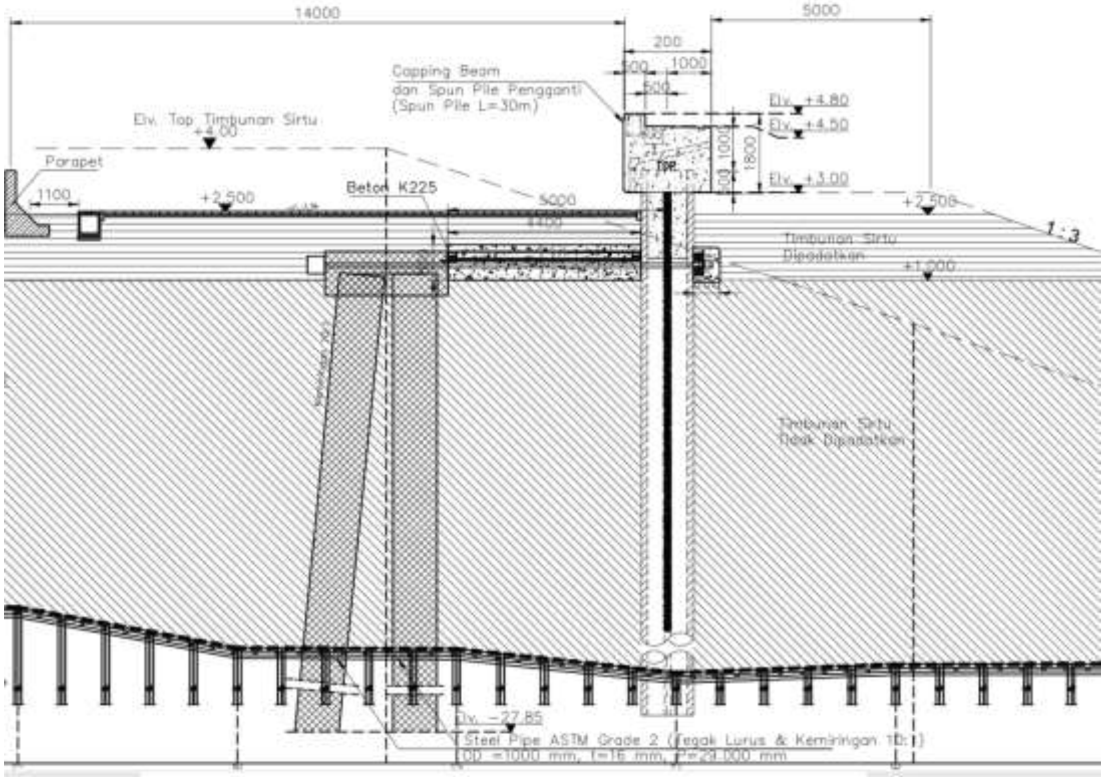

Figure 2. Details of the wall, capping beam, and steel pipe pile in the strengthening system.

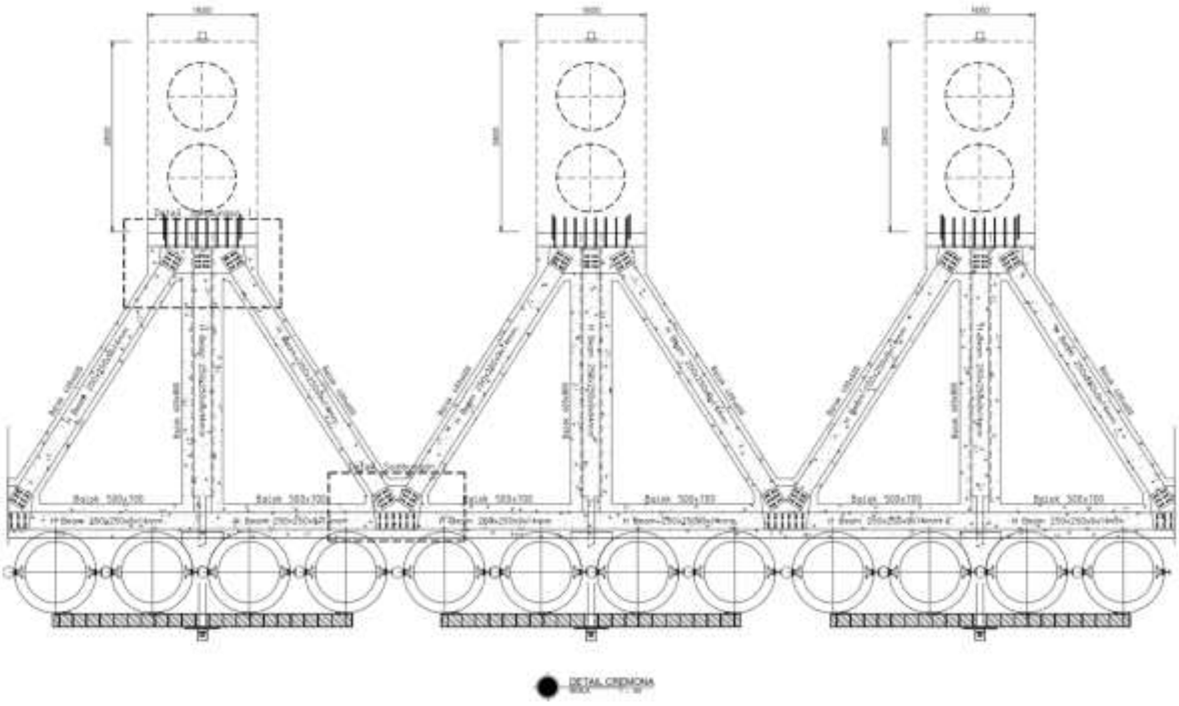

Figure 3. Cremona steel-bracing configuration used in the embankment strengthening system.

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2.2. Global structural modelling and design criteria

The strengthening system was analysed using SAP2000. According to the design documents, structural loads were obtained from PLAXIS 2D output; piles and tiebacks were represented by frame elements; the pilecap was represented by shell elements; and soil interaction was simplified using soil springs. This created a staged analytical workflow in which soil and wall response were evaluated first, after which the resulting geotechnical forces were transferred to the structural model for member-capacity checks.

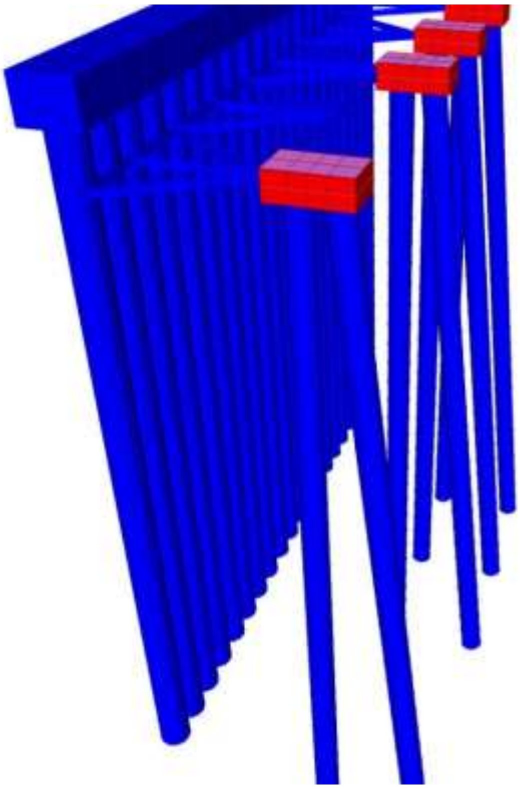

Figure 4. SAP2000 model of the strengthening system.

Table 1. Principal material criteria used in the structural model

Component	Specification	Remarks
Cast-in-place concrete	f'c ≥ 30 MPa	Pilecap/concrete elements
Structural steel sections/plates	ASTM A572 Gr. 50 or SM490 YA/B	Frame members and plates
Structural bolts	ASTM A325	High-tension bolts
Anchor bolts	ASTM A307, minimum fy = 240 MPa	Steel-to-concrete connection
Structural reinforcement	minimum fy = 420 MPa	Deformed bars; SNI 2052:2017

Steel-member checks followed the LRFD approach adopted in the project model, with the requirement that the demand-to-capacity ratio should not exceed 1.0. Accordingly, a unity ratio was used as the acceptance threshold for the global assessment discussed in this paper. Local connection calculations were not reconstructed; the study uses the documented results of the project connection checks.

2.3. Utilization-ratio assessment

For the spun-pile wall, the design report provides axial-force and bending-moment demands together with nominal section capacities. To facilitate comparison, two utilization ratios were calculated as follows:

$$U_N = \frac{N_u}{\emptyset P_n} \quad (1)$$

$$U_M = \frac{M_u}{\emptyset M_n} \quad (2)$$

A value of U < 1.0 indicates that the reported demand remains below the capacity adopted in the design report. These ratios are not intended to replace a full PMM interaction equation; rather, they provide a simple indicator of the utilization level associated with the force components available in the source table.

2.4. Local connection assessment

Two connections were identified as critical: the WF 200×200-to-pilecap connection and the connection between WF 200×200 members within the Cremona frame. The design document states that both connections were modelled using finite-element analysis. Within a CBFEM framework, forces in components such as bolts, plates, welds, anchors, and concrete are obtained from the numerical model and each component is subsequently checked against its design resistance. This approach is appropriate for connection geometries and stress distributions that are difficult to represent with one-dimensional member models (Morido-García & De Santos-Berbel, 2024).

3. Results and Discussion

3.1 Global member performance

The SAP2000 steel check indicated that all steel members in the strengthening system had stress ratios below 1.0. This result shows that the selected profiles remained within the project capacity criterion at the global-model level. Because the structural model received loads derived from PLAXIS 2D, the result also indicates that the transferred geotechnical demand could be accommodated by the frame system at the level of global member verification.

Nevertheless, a satisfactory global member check is not sufficient to demonstrate adequate connection performance. The SAP2000 line-element model represents members through their centrelines and does not capture local stress concentrations, bearing around bolt holes, gusset-plate deformation, or force distribution within bolt groups. Local connection assessment is therefore still necessary. This is consistent with recent studies of bracing connections showing that gusset geometry and bolt configuration can significantly influence stress patterns and failure modes (Shin & Kim, 2025; Xie & Zhang, 2024; Zheng et al., 2026).

3.2 Spun-pile capacity assessment

Table 2. Utilization ratios for the 1200-mm-diameter spun pile

Case	N (kN)	Ncap (kN)	Ratio	M (kNm)	Mcap (kNm)	Ratio
1	1523.292	7215	0.211	0	1200	0
2	1285.447	7215	0.178	1119.726	1200	0.933
3	1162.646	7215	0.161	42.7208	1200	0.036

The highest axial utilization ratio was only 0.211, indicating that axial capacity did not govern the three reported cases. In contrast, Case 2 produced a bending utilization ratio of 0.933. Although this value remains below unity, it indicates that approximately 93% of the bending capacity adopted in the design table was mobilized. Bending demand is therefore the more sensitive parameter for assessing the spun-pile wall in this configuration.

The difference between axial and bending utilization is important for the strengthening system. Lateral earth pressure and reaction from the tieback system tend to generate significant flexural demand in the pile wall, while the axial capacity retains a relatively large margin. Consequently, changes in embankment geometry, ground condition, or tieback stiffness are likely to be reflected more rapidly in bending demand than in axial demand.

The need to interpret spun-pile response on a performance basis is also consistent with Refani et al. (2026), who showed that for spun-pile-supported wharves in marine environments, changes in material condition and loading can influence strain demand, lateral capacity, and performance margins. Although that study considered long-term degradation combined with seismic loading, the distinction between demand and capacity remains relevant to the Muara Baru case: a low axial utilization ratio does not automatically imply a large bending reserve. The bending ratio approaching unity in Case 2 should therefore be regarded as an indicator of greater sensitivity to changes in lateral pressure, tieback stiffness, and soil-reaction distribution.

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3.3 Steel connection assessment

The documented local checks indicate that both connection types satisfied the project criteria. Detail 1 connects the WF 200×200 member to the pilecap, whereas Detail 2 connects WF 200×200 members within the Cremona frame. Figure 6 presents the connection models and the corresponding finite-element stress distributions used in the project assessment.

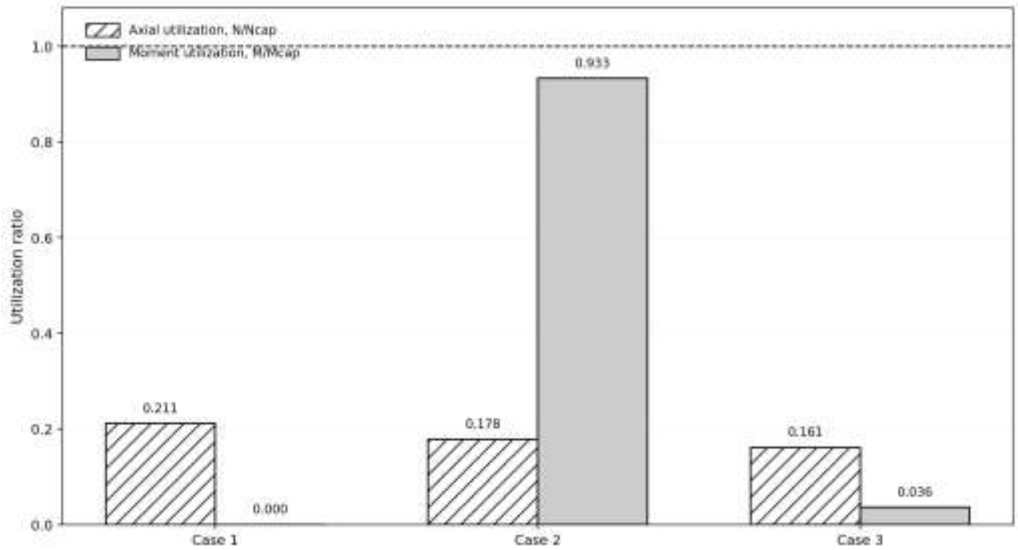

Figure 5. Comparison of axial and bending utilization ratios for the spun pile based on the project capacity-check table.

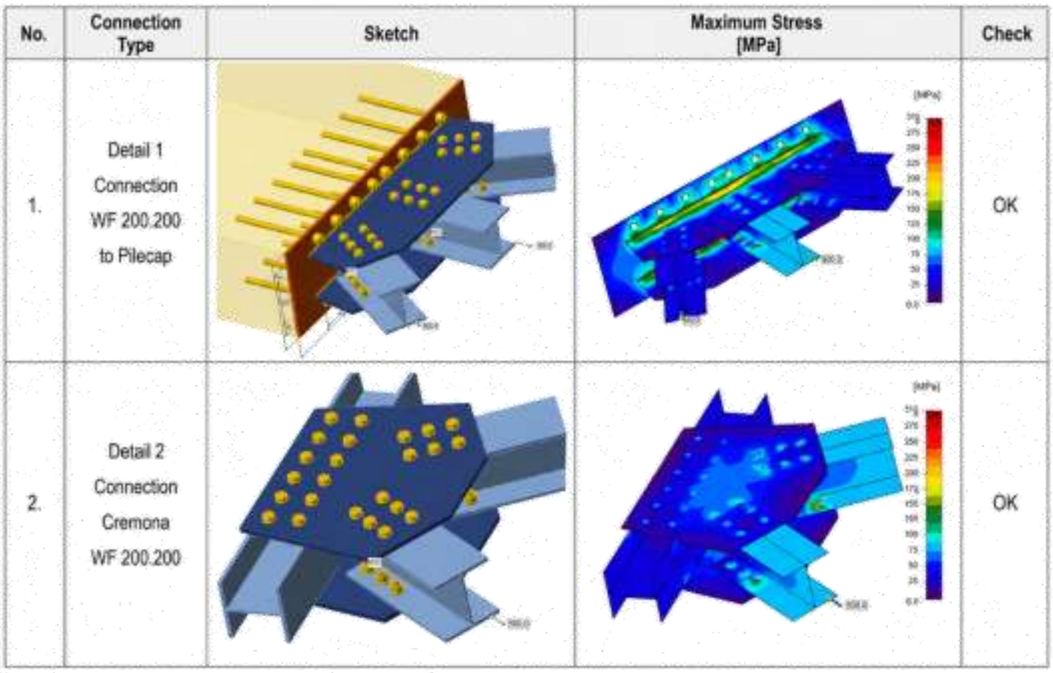

Figure 6. Finite-element assessment of the two principal connections in the strengthening system

Detail 1 uses 48 M20 bolts and 16 M28 anchors with an embedment depth of 400 mm. The relatively large number of fasteners reflects the need to distribute the load transferred from the bracing system to the pilecap and to avoid concentrating demand in a small anchor group. Detail 2 uses 64 M20 bolts to connect the Cremona-frame members through a gusset plate. Mechanically, this configuration is consistent with bracing connections that require force distribution over a sufficiently large plate area. Experimental and numerical studies of gusset plates have shown that stress concentration, net-section rupture, block shear, and local buckling can become critical when geometry or restraint is inadequate (Shin & Kim, 2025; Song et al., 2023; Zhang et al., 2023; Zheng et al., 2026). For the pilecap connection, recent base-plate studies also reinforce the importance of plate thickness, anchor arrangement, and stiffening in controlling force distribution and connection stiffness (Nawar et al., 2024; Jiang & Billah, 2025; Khani et al., 2025).

The reported "OK" status of the connection models indicates that the selected configurations satisfied the project checks; however, the interpretation in this paper remains limited by the available data. The source report does not provide complete numerical tables for bolt forces, plate deformation, plastic strain, weld utilization, or individual component utilization. The colour contours are therefore not interpreted as specific capacity values. Local conclusions are limited to the documented check status and connection configuration. This limitation is important because recent studies of base-plate connections show that plate thickness, anchor diameter and layout, and stiffener configuration can substantially alter stiffness, moment capacity, anchor-force distribution, and failure mode (Nawar et al., 2024; Jiang & Billah, 2025; Khani et al., 2025).

Table 3. Summary of steel-connection details

Parameter	Detail 1: WF 200×200-to-pilecap	Detail 2: Cremona WF 200×200
Steel plate grade	ASTM A572 Gr. 50 / SM490 YA/B	ASTM A572 Gr. 50 / SM490 YA/B
Bolt grade	ASTM A325	ASTM A325
Anchor grade	ASTM A307	–
Gusset plate thickness	16 mm	16 mm
Base-plate thickness	16 mm	16 mm
Bolts	M20; 48 points	M20; 64 points
Anchors	M28; 16 points	–
Anchor embedment depth	400 mm	–
Check result	OK	OK

3.4 Design implications for coastal-structure rehabilitation

The case study indicates that strengthening assessment is more effective when performed in stages. The first stage evaluates the global response of the structural system to loads derived from the geotechnical analysis. The second stage compares member demand with member capacity. The third stage transfers the demand to local connection models to verify continuity of the load path. This sequence reduces the risk of declaring the principal profiles adequate while leaving connection behaviour or force transfer to the foundation insufficiently verified.

For steel-braced embankment rehabilitation, three outputs should be interpreted together: (1) the global member stress ratio, (2) wall/pile utilization, and (3) local connection response. In the Muara Baru case, the frame members satisfied ratios below 1.0, the spun pile retained a large axial-capacity margin, while the bending utilization ratio approached unity in one case. These results identify pile-wall bending and local force-transfer details as the two aspects that warrant particular attention when geometry or loading conditions change.

4. Conclusion

The global SAP2000 model, which received loads from PLAXIS 2D analysis, showed that all steel members in the strengthening system had stress ratios below 1.0 in accordance with the project acceptance criterion. For the 1200-mm-diameter spun pile, the maximum axial utilization ratio was 0.211, whereas the maximum bending utilization ratio was 0.933 based on the project capacity-check table. The available data therefore indicate that flexure is more critical than axial demand for this configuration. The two principal connections, namely the WF 200×200-to-pilecap connection and the WF 200×200 Cremona-frame connection, were reported to satisfy the IDEA StatiCa finite-element checks. The connection configuration uses 16-mm gusset and base plates, ASTM A325 bolts, and ASTM A307 anchors for the pilecap connection. A staged assessment from global response to member capacity and finally to local connection behaviour provides a practical framework for verifying the structural load path in coastal embankment strengthening.

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