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Conceptual Design of Suspended Structural Systems

by

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April 22, 1998

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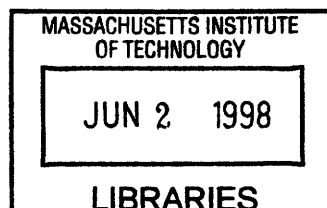
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Abstract

This thesis investigates the use of suspended structural systems. This is done by performing a conceptual design for the proposed new Civil and Environmental Engineering building at the Massachusetts Institute of Technology. It begins with site considerations and owner requirements and progresses through the concept development and design development.

Preliminary member sizes are determined. The majority of these are calculated using computer analysis, but hand calculations are included as well. In addition, a 3-dimensional model is created in SAP 2000. This model was used to analyze the impact of dynamic loading on suspended systems.

AutoCAD is used to create 2- dimensional architectural and structural drawings. It is also used to create 3-dimensional architectural drawings. AutoVISION is used to apply materials to the surfaces of the drawings and render images. AutoVISION is also used to create a simulated 'fly-by' and 'walk-through.'

The issue of constructibility is addressed and erection techniques are proposed. The topic of scheduling construction phases is briefly discussed as well.

The thesis includes a rough cost analysis with suggested refinements to enhance the design and reduce the cost.

Thesis Supervisor: Professor Jerome Connor

Acknowledgments

I would like to express my gratitude and admiration of my advisor, Jerome Connor. His knowledge and guidance taught me an immense amount about the dynamics of structural systems. His ability to remember even the minutest details and his willingness to help contributed so much to this thesis.

I am grateful to Frank Heger for choosing to spend some of his time talking to the other High Performance Structures students and myself. His passion for the field was one of my strongest motivators.

I would like to thank Jack Nevins for helping me by providing structural drawings for the Rotch Library at MIT. It is one of the few suspended buildings around and being able to look through the drawings was a huge help. It is amazing how many students obtain a degree in structural engineering without ever seeing structural drawings.

Richard Henige offered his knowledge and advice in resolving the numerous design problems associated with the exterior columns. I appreciate the time Richard spent with me.

I would like to thank Robert McNamara of McNamara/Salvia for his direction in the dynamic modeling using SAP 2000 as well as his advice on the use of linear viscous dampers.

Of course, I can not forget my group members: Matt Kmetz, Belden Nago, Vitto Agnesi and Casandra Strudwick. It was all of us working together that came up with this whole idea. Matt was an integral part of the manipulation of the immense amount of data obtained from the dynamic analysis.

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1 Introduction

1.1 Purpose

This report takes a broad look at the practicality of suspended structural systems. To accurately investigate the practicality, an actual building needs to be looked at. The building that is investigated is a new facility for the Civil and Environmental Engineering Department for the Massachusetts Institute of Technology. The building is intended to occupy the current location of buildings 45 and 48. The conceptual design should be defined enough to make an educated decision on whether or not to pursue the idea further. If the conceptual design were deemed feasible, the next stage would be to refine the owner's requirements and desires and then progress into a final design.

1.2 Scope

The conceptual design of this suspended structure includes first cut member sizes. The building is analyzed both statically and dynamically to be built in eastern Massachusetts. The foundation, cladding, partitions and roof are not included but are discussed to some extent in the Master of Engineering High Performance Structures Group Report. [10] The design includes a rough cost estimate. The report includes a 3-dimensional computer model of the building including a short movie.

2 Facility Requirements

2.1 Size

The Civil and Environmental Engineering Department at the Massachusetts Institute of Technology set guidelines for a minimum of 110,000 square feet of net area. The initial use of the building is intended to be office and laboratory space. To accommodate this and other future flexibility issues, the design requires large clear spaces between floors. The desire for a flexible structure also implies an open floor space. The optimal solution is a structural system with no interior columns.

2.2 Loads

In an attempt to adhere to the flexibility guidelines of MIT's physical plant, all floors are required to support a 200-psf live load. [2] This allows for the unrestricted use of all floor space, i.e. laboratory and office space can be allocated as the department deems fit rather than in accordance with the design. During the refining stages of the final design, this live load could be reduced to values that are more realistic in order to lower the cost.

The snow and wind loads were obtained from the Massachusetts State Building Code 780 CMR. [8] The earthquake loads were applied in accordance with the Building Code, but were applied dynamically, in addition to statically, for a more accurate response. SAP 2000 was used to compute the time history response of the building to scaled versions of the El Centro and Northridge earthquakes.

2.3 Aesthetics

The importance of the building as a symbol of the engineering capacity of the Massachusetts Institute of Technology and the Civil and Environmental Engineering Department in particular demands an innovative design. To accomplish this, the structural design precedes the architectural design. The architectural aesthetics of the building are considered, but in a way that highlights the structural system. The objective

is to create a unique and innovative facility, which will be a showcase for the exemplary department and institution that it houses.

2.4 Site Characteristics

Layout

The proposed site is on the northeast corner of Main and Vassar streets and can be seen in Figure 2.1. The Institute intends to demolish the existing Civil and Environmental Building (Bldg. 48) and Animal Sciences Building (Bldg. 45). The dimensions of the cleared site are approximately 530 feet long and 115 feet wide. (Site Maps Appendix-A) The site is bordered on the north by a non-mainline railway. The Cyclotron Building (Bldg. 44) borders the site on the west.

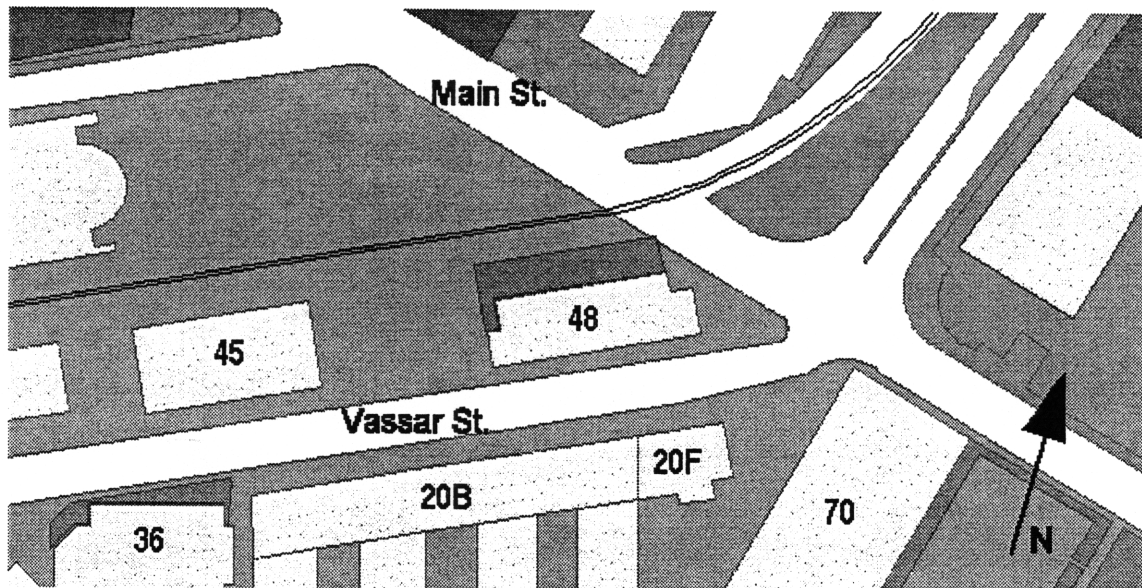


Figure 2.1 Northeast campus map

Environment

The close proximity to the railroad causes significant disturbance in the existing buildings. The passing trains cause the building to shake and have a large acoustic effect as well. The vibrations induced due to automobile traffic over the uneven road are also a problem. Both of these problems are not desirable and the effect is to be minimized in the new facility. The new facility is intended to house sophisticated monitoring equipment.

3 *Concept Development*

3.1 *Site Layout*

The nature of the proposed site lends itself to a long narrow building along Vassar Street. (Site Maps Appendix-A) The nearby proximity of the railway suggests that the building be located as close to Vassar Street as possible to minimize the noise and vibration absorbed from the passing trains. The Master Plan [2] of the Massachusetts Institute of Technology indicates the institution's desire to shift the center of campus toward the corner of Main and Vassar Streets. The institute has already procured the services of Frank O. Ghery for the design of the new Electrical Engineering and Computer Science Department building. This new building is to be located on the southwest corner of Main and Vassar Streets. The selection of a high profile architect supports the Institute's Master Plan. The potential of this corner to be a major entrance to the Massachusetts Institute of Technology makes it a prime location to display a unique and daring civil-engineered facility.

3.2 *Architectural Layout*

To make the building a hallmark of the Massachusetts Institute of Technology and the Civil and Environmental Engineering Department, the latest technologies and practices in structural damping are incorporated into a suspended structure. The structural columns of the building are offset 10 feet from the main floor space. This allows for a 100 foot tall glass atrium to envelop the building. In addition, to take advantage of the building's location, the building design incorporates a tensile roof structure over a reduced height section at the east end of the building. Coupling the unique roof structure with the outer façade of mildly tinted glass is intended to draw passersby's eyes to the exposed structure of the building. Adding to this will be the absence of columns between the bottom floor of the suspended structure and the ground. (Structural Drawings Appendix-D)

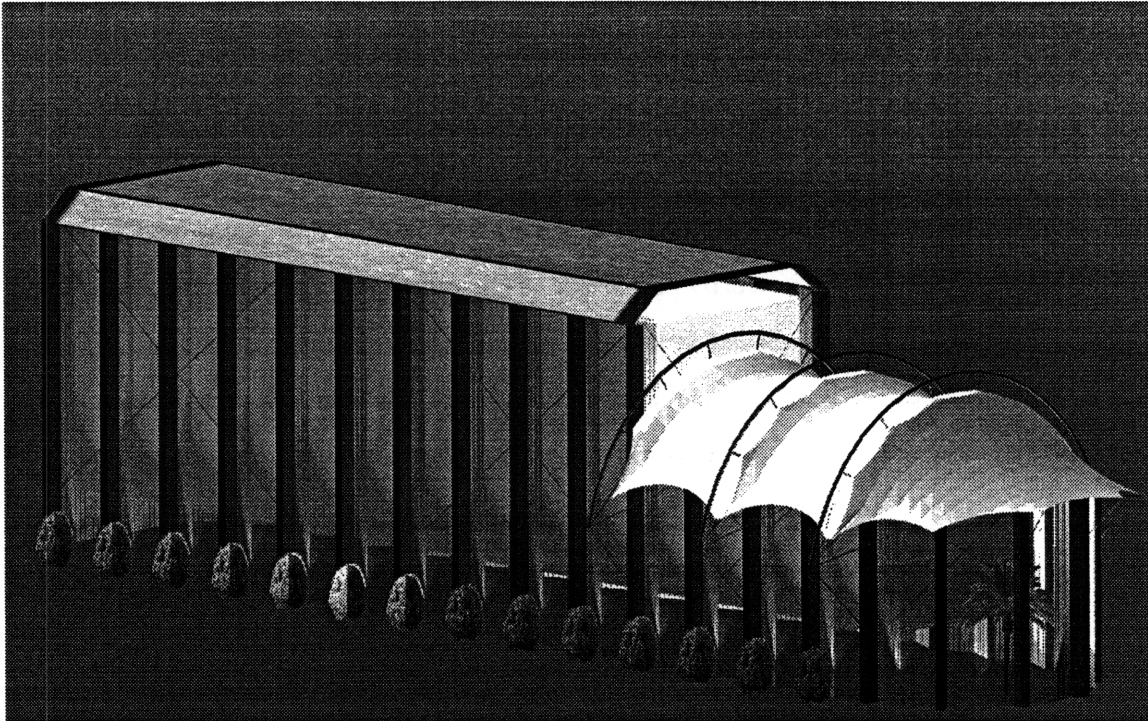


Figure 3.1 Southeast isometric rendering.

The design requirement of no interior columns and a high design load limits the length of a structural bay to a maximum of 60 feet. To allow for the flexibility of future use, the floor-to-floor height is 20 feet. The Cambridge City Code restricts new construction to a height of 120 feet, which forces the building to extend towards the west. For the building to have a minimum of 110 thousand square feet of space, the structure, including the atriums, will have a 80 foot by 400 foot footprint. This space is spread out over the basement, ground floor and the four suspended floors. The first four bays, or 100 feet, of the building on the east end will only support two hung floors and will only rise 70 feet. This is being done to make to make the tension roof structure more visible from the ground. The reduction of the height also provides a unique architectural break to the building.

Description

The exterior columns are spaced at 25 foot on center. They rise 100 feet unbraced to a deep truss. The truss spans 80 feet across to another column on the other side of the building. The main building structure is 60 feet wide and is supported only at the edges by the truss above. Each floor is supported by a cable connected in series to the floor above it and eventually to the truss. (Architectural Drawings Appendix-C) For the first five frames on the east end, the columns only rise 60 feet to the truss. The ground floor and basement are an independent structure. The exterior columns support an outer façade of glass that, in addition to creating a tall atrium, functions as the building envelope. The wind load travels directly down the exterior columns, thus bypassing the suspended structure.

Advantages

The most economical use of the building would be to locate all of the wet laboratories as well as other labs not sensitive to mild vibrations in the basement and on the first floor. The suspended structure could then house the classrooms, offices and laboratories sensitive to vibrations. In this structure, measures could be taken to isolate the necessary facilities from even the minutest vibrations. This is practical because numerous Civil Engineering laboratories are sources of vibration themselves.

The outer façade of glass keeps the wind from interacting with the suspended structure that has a modal frequency that is similar to that of the wind. The outside frame is also a natural isolation system to earthquake loads. This is because of its flexibility. The columns and truss alone have a period of about 1.5 seconds.

4 Design Development

4.1 Structural Layout

The building is supported by structural frames spaced every 25 feet on center along the long axis of the building. Each structural bay consists of two columns with a 100 foot unbraced length supporting a 80 foot long deep truss. The truss is 10 feet deep and will be covered by a corrugated steel deck roof. The bottoms of the trusses are flush with a concrete slab that will support the majority of the mechanical equipment for the building. (Structural Drawings Appendix-D) Ten feet inside the building, two high-strength steel cables are affixed to the truss and connected to the top floor of the building. The cables connect to deep steel girders supporting the individual floors. Each girder spans 60 feet across the building to the connection with a cable on the opposite side. The girders support steel beams spaced every 10 feet on center. These beams span 25 feet between girders and support the floor. The floor consists of a rolled steel deck with a concrete slab. (Structural Drawings Appendix-D)

An expansion joint is located at the interface between the five and three story sections. The overall length of the building borders on the professional limit of spans without control joints. In addition, the different surface areas and masses of the two sections will cause an unsymmetrical response to lateral loading.

4.2 Gravity Load Member Design

All members are design in accordance with the Load and Resistance Factor Design Manual 2nd Edition. [9] An educational version of SAP 2000 was used for analysis and design of the wide flange and steel cable sections. All steel sections are 50ksi steel. The cables have a design strength of 120~130ksi.

Slab

The beam spacing of 10 feet and the girder spacing of 25 feet allows one-way slab design methods to be applied. The magnitude of the live load requires the slab design to be composite. Composite slab-deck interaction is achieved by using a steel deck that is stamped with teeth to provide a bearing surface between the two materials. Another advantage of composite construction is the deck serves as the formwork for the slab during placement. This lowers construction costs. After curing and upon loading the concrete is stressed in compression and the steel deck serves as the positive moment reinforcement. In consideration of the load path through the system, the use of lightweight concrete deemed itself practical. [1] For the given loading and span, the specifications suggest a 5 inch lightweight concrete slab poured over a 3 inch, 16-gauge, corrugated steel deck (3VLI16). This specification allows for a 202-lbs/ft² live load. The dead weight of the slab and deck is only 34 lb/ft². If normal weight concrete were used, an additional ½ inch of concrete would be required and the dead load would be increased to 50 lb/ft². This justifies the additional material cost of lightweight concrete.

Beams

The beams support the distributed loads of the slab and carry it to the girders. The distributed dead load is .34 kips/ft from the slab plus its self-weight. The live load is 2 kips/ft. The beams are spaced every 10 feet and span 25 feet to the girders. To minimize the negative moment effect on the slab, the beams-girder connection are pinned. By only carrying shear, the beam will not have a negative moment region. In addition, this eliminates eccentric girder loading along the exterior of the building. Computer aided structural analysis of the member yields a W18x50 rolled steel section.

Girders

The girders support the concentrated load from the beams every 10 feet and carry it to the steel cables on the outside of the hung structure. The connection to the cables is also a pinned connection because cables are unable to carry moment. Computer aided structural analysis of the member yields a W36x393 rolled steel section. The controlling factor for

the design of this member is the deflection. The allowable live load deflection is 2 inches ($l/360$) and the allowable service load deflection is 3 inches ($l/240$).

Composite action in the girder was considered, but the use turned out to be less efficient than non-composite design. According to structural mechanics, the slab must be thickened to account for the additional stresses. This is because the slab and girder span the same direction and their stresses add. In order to use composite girder design, the slab would need to be thickened a few inches. The added dead weight did not justify the cost savings gained from using a smaller steel member. However, in professional practice, this adding of stresses is neglected and composite action is used.

Interior Cables

The girders are supported at the ends by high strength steel cables. The design of the cables was estimated by using an average strength for the steel based on the gross area. Typical steel cables have an average allowable stress of 120 ~ 130 kips/in². The design of the cables is based on a 120kips/in² allowable stress to be conservative. Each floor adds an additional 255 kips of factored loads.

2nd Story Cable

$$\frac{255kips}{120ksi} = 2.125in^2 \quad D = 2\sqrt{\frac{2.125}{\pi}} = 1.65in \quad (4-1)$$

Therefore, the second story cables are designed to be 1 3/4 inches in diameter. The calculations for the remaining floors can be found in the appendix. (Gravity Cable Design Appendix-E) The Massachusetts State Building Code allows for a reduction to .8 of the live load for the cable members. A specific type of cable is not specified, but there is no need for flexibility, which would allow for a less expensive braid of cable.

Truss

The truss is a traditional design with the largest member sizes limited to 24 inches in depth. (Structural Drawings Appendix-D) The truss was modeled in SAP 2000. The

program output and the design sections are located in the appendix. (SAP 2000 Output Appendix-I) The four trusses that span over the shorter section of the building are of the same design to ease constructibility. The Massachusetts State Building Code allows for a reduction to .8 of the live load for the truss members.

Exterior Columns

The 100 foot unbraced length of the exterior columns proposed a difficult design problem. The effective length multiplier makes it imperative that the bases of the columns be classified as fixed and carry moments. The scenario of a fixed base and a free top requires an effective length multiplier (K-factor) of 2.0. The maximum factored axial load after the allowable .8 reduction of the live load is 1170 kips. (SAP 2000 Output Appendix-I) For aesthetic purposes, a box section was chosen.

Tall Columns

try 20" x 20" -³/₈" thick

$$\frac{KL}{R} = \frac{(2.0)(100\text{ft})}{7.99\text{in}} = 25.0 \quad \text{from Table E-1 (LRFD vol.1)} \quad \phi_c F_{cr} = 40.60 \text{ ksi} \quad (4-2)$$

$$\frac{\text{Load}}{\phi_c F_{cr}} = \frac{1170\text{kips}}{40.60\text{ksi}} = 28.8\text{in}^2 \quad \text{area of section} = 29.1\text{in}^2 \quad \text{O.K} \quad (4-3)$$

The option of using wide flange sections was investigated, but proved aesthetically unpleasing. However, there is significant cost savings in avoid the use of box sections. Another advantage of the box section is the ability to fill them with concrete. This reduces the likelihood of local buckling. While not only adding a factor of safety to the axial load design, the additional weight of the concrete decreases the soil uplift and reduces the foundation cost.

4.3 Seismic Design

Static Analysis

Exterior Lateral Bracing

The large unbraced length of the exterior columns creates numerous interesting design considerations. The base of the column is designed as a fixed connection while the top of the column is classified as a free end. The truss supporting the interior structure bears at the tip of the cantilevered column. A pinned connection eliminates a moment at the tip. The lateral loads cause a deflection that is compounded by a non-linear “P-delta effect.” This occurs when the initial deflection causes an eccentricity, which creates a moment. This additional moment increases the deflection. This process repeats itself until the changes become relatively negligible.

The controlling lateral loading is the earthquake loading. Using the method prescribed by the Massachusetts State Building Code CMR 780, the lateral load applied at the top of each column was determined. The interior building acts a pendulum, with the only lateral stiffness coming from the interior cross bracing. Therefore, the building will have the same characteristics in either direction. This requires the exterior columns to take the same lateral load in either the transverse or longitudinal direction. This loading is most efficiently dealt with by using box sections for the column members. The member dimensions are unpractical if the column is designed to take all of the lateral loads. For this reason, steel cable bracing is needed. A problem with lateral bracing is its interference with the useable space as well as aesthetic impacts. However, an efficient bracing scheme that does not infringe on the useable space is possible

The layout of the bracing (Architectural Drawings Appendix-C) minimizes the interference with the interior space. The cables only cross through the building at the joint between the short and tall portions of the building. The cables along the building façade are attached to the exterior columns, which are not required to be fireproofed. All of the cables attach to the columns at 20 feet above the ground level. This eliminates the interference with the first floor walking space. It also limits the interference where the

cables cross through the building. They do not cross the expected hallway location. The largest cable is only 1-½ inches in diameter, which means that they will be almost invisible to the casual onlooker. The columns easily carry the shear load applied by the cables at the 20-foot elevation with no significant effects. They also bear the wind loads with no problems.

Internal Lateral Bracing

The suspended structure requires additional floor-to-floor stiffness in the form of steel bracing members. This is because the members attaching the floors to the roof trusses are steel cables and therefore do not provide the required level of stiffness. The amount of steel required per floor was calculated by first using the ASCE Minimum Design Loads for Buildings and Other Structures to calculate the effective, statically applied earthquake loads at each level of the building. [8] Enough bracing was then installed to carry these forces. The braces are located along the outside of the building at the corners. They will most likely be enclosed in the wall around the elevator shafts. Double angle sections were selected. The maximum shear between floors for an entire floor was calculated to be 163.5 kips. Due to the unknown effects of the dampers at the base of the building, all floors are design to carry this maximum shear. Assuming the braces only act in tension, there are four braces per floor. The shear load is carried by two L2x2x³/₈. Due to the symmetry of the building and loading possibilities, the same bracing is adequate for either direction. For constructibility considerations, the same bracing is used in the short portion of the building.

The equations and constants required to calculate the lateral seismic force induced at the base of the structure and the calculations for the lateral bracing cable design can be found in Appendix-G.

Dynamic Modeling and Analysis

The full non-linear version of SAP 2000 was used to create a 3-dimensional model of a typical frame. The complexity of a suspended structure prohibits being able to generalize the building as a shear member. To accurately predict the behavior of the building under

dynamic loads, an entire planar frame was modeled. The building is assumed to react similarly in the x- and y-direction and therefor, is only analyzed in the x-direction. This model is shown in Appendix-I.

The model was subjected to the dynamic loading of two earthquakes: El Centro and Northridge. The time histories of the two earthquakes are shown in Figure 4.1 and Figure 4.2 respectively. These earthquakes have been scaled to the Massachusetts State Code for eastern Massachusetts. The earthquakes were then applied to the building individually along the x-direction. The building is assumed to have 5% natural damping inherent in the structure.

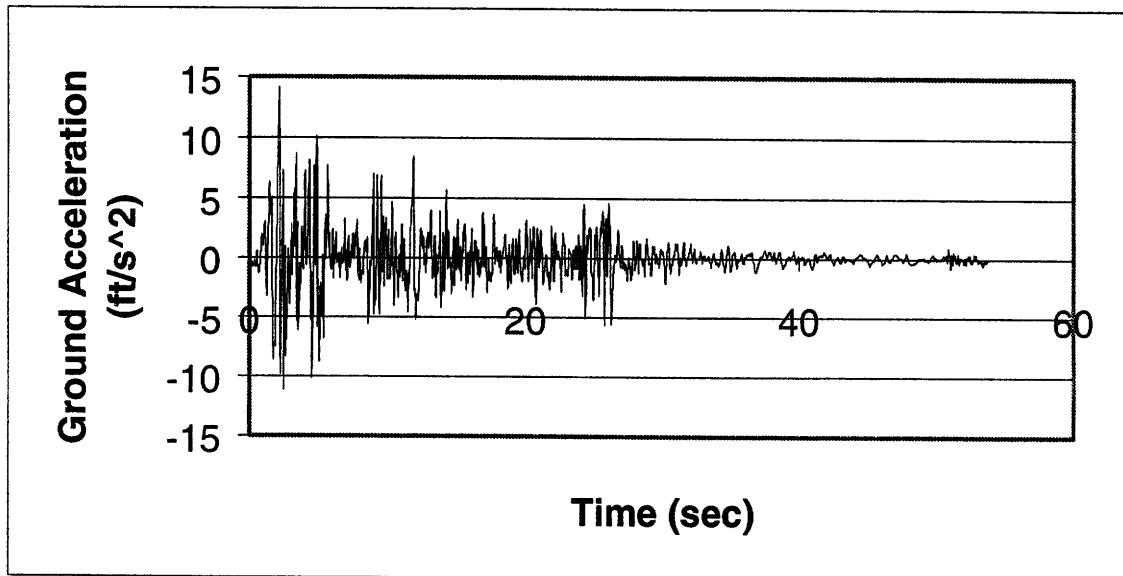


Figure 4.1 El Centro ground acceleration versus time.

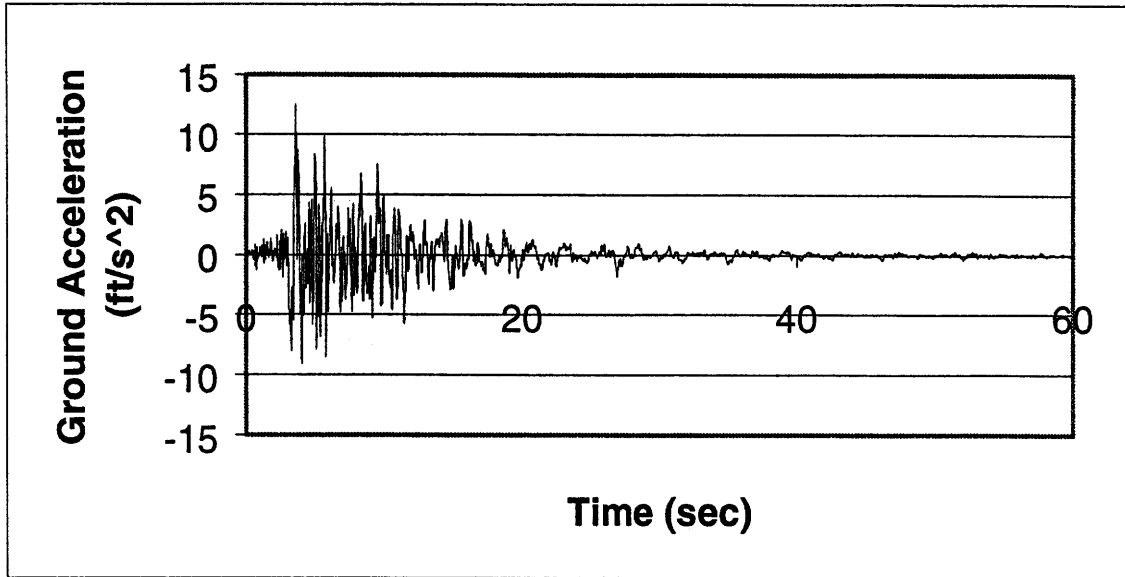


Figure 4.2 Northridge ground acceleration versus time

Figure 4.3 and Figure 4.4 show the base shear due to the earthquake loadings of El Centro and Northridge, respectively. These values are quite high and cause unnecessary strain on the foundation. It is obvious that improvements can be made through the addition of a passive damping system.

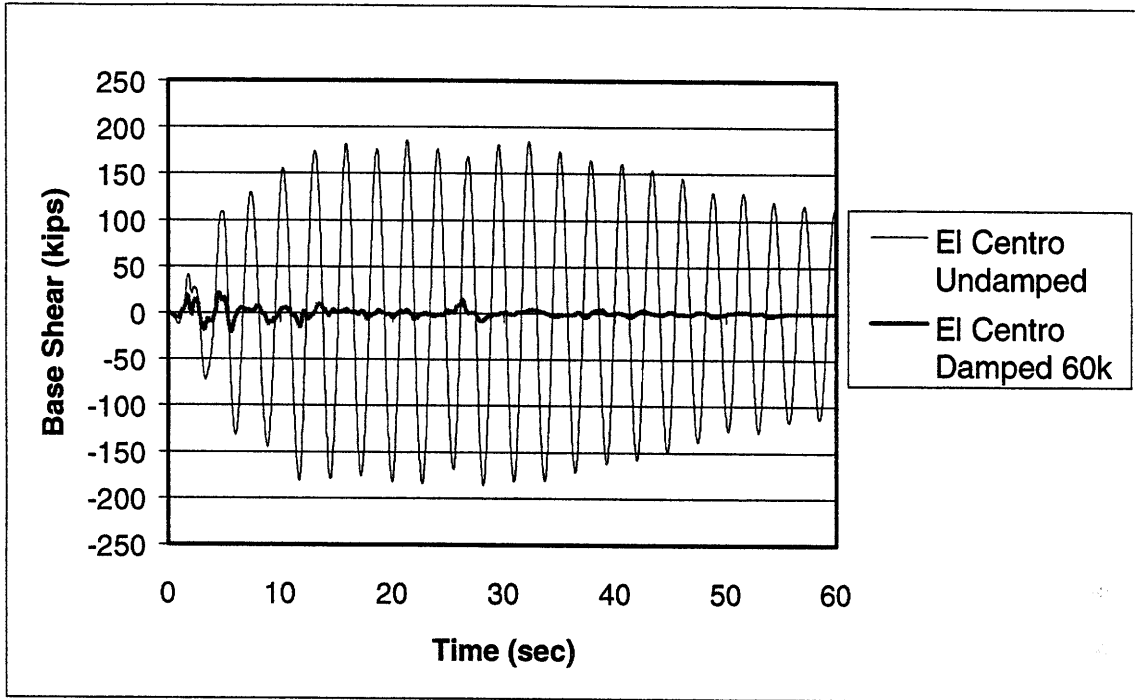


Figure 4.3 Base shear due to El Centro

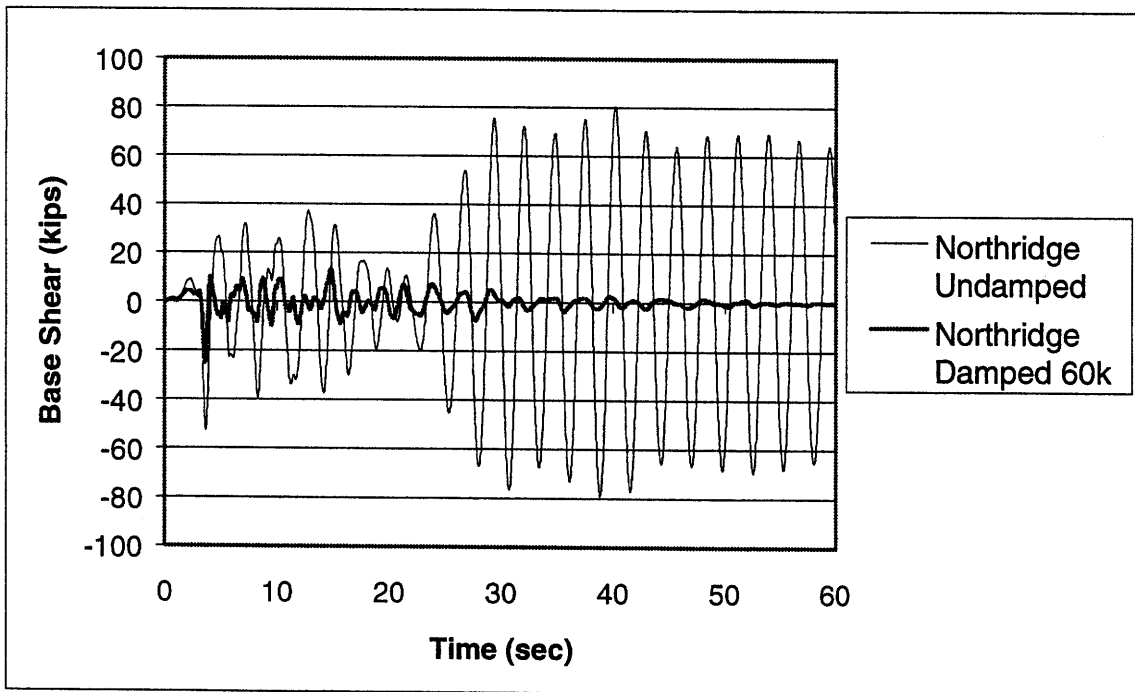


Figure 4.4 Base shear due to Northridge

Before devising a damping scheme, one must understand the behavior of a damper. Linear viscous dampers produce a resisting force proportional to the velocity and relative displacement between two points. For this type of damper to be effective, the dampers need to be placed in locations that have large relative motion. The behavior of a suspended structure makes them the ideal choice. The first mode is the most critical and conveniently, one of the largest motion occurs at the base of the interior structure. This is very close to the ground, which is considered to have no motion. Therefore, the relative motion is very high. The optimal solution is to attach the dampers to the ground and to the base of the interior structure because the movement is a maximum of the first mode and all of the motion is relative.

In addition to the undamped base shear, Figure 4.3 and Figure 4.4 also have curves representing a damping scenario. The 60k represents the equivalent damping per damper per frame. Thus, a 60k damping scheme for the tall portion of the building would require two 300-kip dampers in each of the two end bays. The dramatic decrease in the base shear shows the impact of the addition of a damping system.

Once the significance of a damping system in a suspended structure is realized, the amount necessary needs to be determined. Figure 4.5 and Figure 4.6 show time history traces of a node at the base of the interior structure due to the Northridge and El Centro earthquakes. The El Centro loading causes a slightly larger response. This is related to the frequency content of the earthquake loading. Without any damping, the same joint displaces 24.4 inches. With the addition of four 150 kip dampers, this displacement is reduced to 4.6 inches. Increasing the damping by using 300 kip dampers instead, the displacement is reduced even more to 2.8 inches.

The behavior of the suspended structure is unique in that the displacement at the top is almost identical to the displacement at the base of the suspended portion. (SAP 2000 Output Appendix-I) The benefit of this is no location in the building incurs a significant shear deformation. With the 60k damping scheme, the only location that experiences any significant shear deformation (γ) is the outer column from the base to the tip. This

displacement is 3 inches. If the allowable inter-story drift is limited to 3.3 inches ($l/360$), then the 60k scheme is satisfactory.

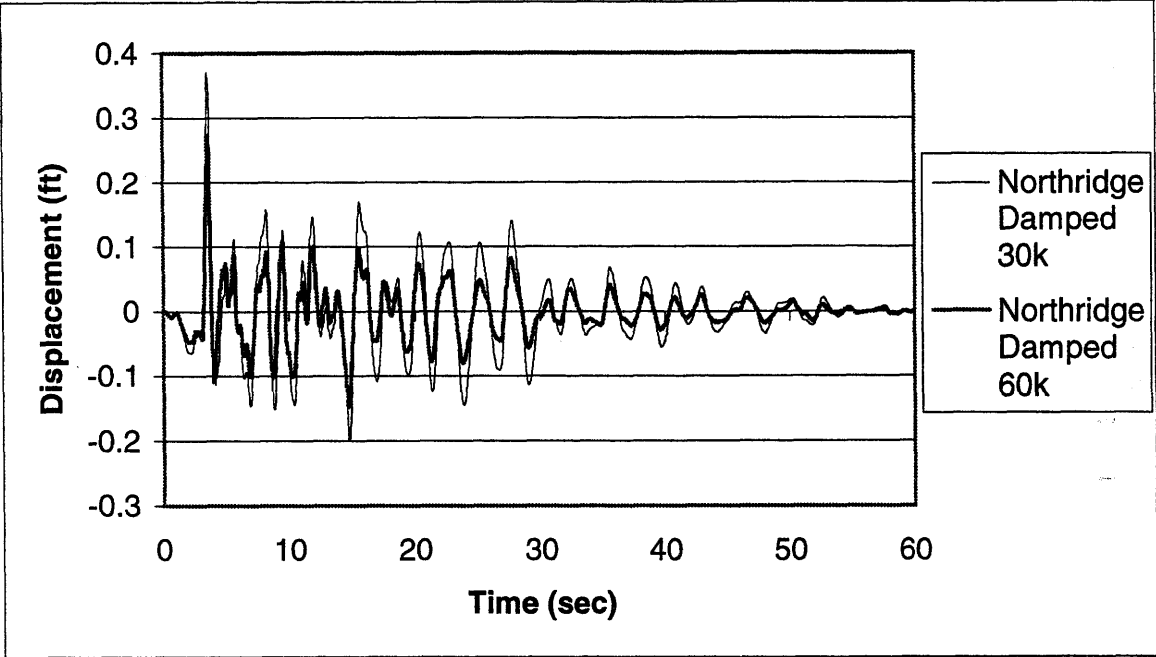


Figure 4.5 Damped displacements at the top of the columns due to Northridge

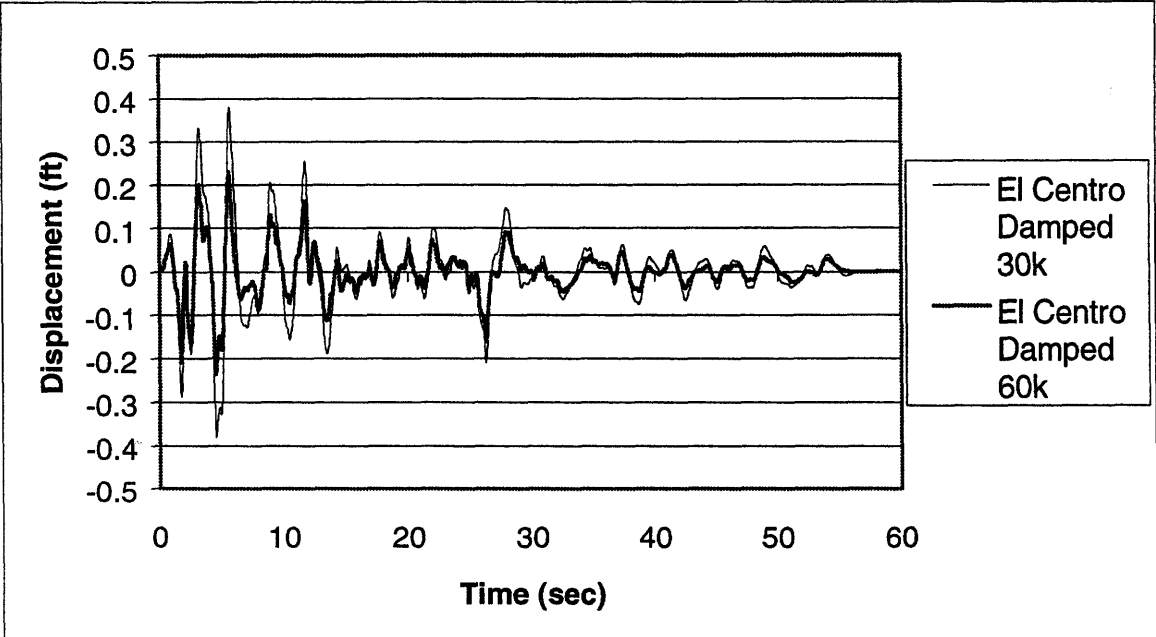


Figure 4.6 Damped displacements at the top of the columns due to El Centro

The response of the three-story portion is similar but the magnitude is reduced. Due to time constraints, an analysis of the short portion was not performed. For the design, a conservative assumption is made that would require the same damping in the short section. This is over designed, but can be refined in the final design.

4.4 Constructibility Considerations

The suspended structure poses numerous constructibility problems, but also affords the opportunity for cost and time savings. The break in the building between the taller and shorter sections allows the structure to be erected in two phases. Building 45 could be demolished and the tall portion of the new structure erected. Then the offices and laboratories from building 48 could be relocated into the new facility. This eliminates the need for temporary space to house building 48 occupants during construction.

The high water table and associated uplift on the structure can be mitigated by only erecting the outer foundation wall that supports the exterior columns. The majority of the soil could be left unexcavated. After the majority of the building has been erected and a significant dead load is present, the basement could be excavated.

In addition to the geotechnical benefits, more innovative erection techniques can be implemented. The floors, which already have to be erected in a top down fashion, could be erected on the ground. The majority of the steel members are uniform and all of the connections are simple bolted shear connections. This minimizes the need for field welding and increases erection time. In addition, working at grade increases erection speed and reduces the likelihood of injuries. Once a floor grid has been completed, it can be raised up and attached. At this point, the floor slab could be pour while erection of the next floor could have already begun.

5 Cost Estimate

5.1 Existing

The unit costs listed in the 1996 edition of Means Building Construction Cost Data were used to determine the material cost estimate for the structural system. A preliminary unit cost for structural steel members, which included the cost of connections and erection, is used. Other material items included square structural tubing, metal decking, welded wire fabric, and structural concrete. The total premium material cost without a damping system is \$2,925,000. An itemized list of the costs can be found in Appendix-J. The addition of the four 300-kip dampers and four 150-kip dampers increases the cost by \$80,000. [6] This cost does not include the foundation, which would add an additional \$1.2 million. The tension fabric roof would also add an additional \$500,000. These items bring the overall building cost to approximately \$4.7 million.

5.2 Proposed Improvements

The most effective way of reducing the cost of the structural system is to refine the live load requirements. Even under extreme conditions, the building will not be subjected to the design live loads. A final design would investigate possible future uses and lower the live load requirement accordingly.

Looking at the spreadsheet in Appendix-J, an obvious source of added expense are the exterior columns. Tube sections are extremely expensive in comparison to rolled wide flange sections. Although they are aesthetically pleasing, other alternatives exist. The most practical solution would be to use rolled sections and then encase them in concrete. An equivalent wide flange section is a W30X173. Use of this section would reduce the building cost by approximately \$200,000

The use of composite action in the girder would also allow for substantial savings. Composite design would allow for a much lighter beam that would lower material cost.

6 Recommendation

6.1 Justify

The choice to use a suspended structural system in lieu of a traditional system is beneficial for numerous reasons. One of the most important reasons is the innovative and daring visible structure and its influence on the reputation of the Civil and Environmental Engineering Department of the Massachusetts Institute of Technology. In addition, the ease of incorporating a damping system makes the suspended structural system very efficient.

6.2 Proposed Improvements

The design could be improved and made more efficient by using composite action in the design of the steel members. In addition, increasing the offset of the exterior columns would magnify the aesthetic effect of the surrounding atrium. This possibility was not investigated because of site limitations. Another obvious refinement would be to lower the design live load. The current, 200-psf live load is too large to be counted on every floor.

7 *References*

7.1 *Bibliography*

1. Nucor Corporation. Vulcraft Steel and Roof Deck Manual
2. Wallace, Floys, Associates, Inc. *Main Campus Northeast Sector Master Plan of MIT*. 1989.
3. Connor, J. J. Introduction to Motion Based Design. Computational Mechanics Publications. 1996. pp. 83.
4. Taylor Devices, Inc. Earthquake Protective Systems for Buildings, Bridges, and Other Structures. 1996.
5. Computers and Structures, Inc. SAP2000 Tutorial Manual. 1997. pp. 18.
6. Rasmussen, Eric. "Dampers Hold Sway." ASCE Civil Engineering, Vol. 67, Number 3, pp. 40-43. March 1997.
7. American Institute of Steel Construction. Manual of Steel Construction—Allowable Stress Design. Ninth Edition. 1989.
8. The Massachusetts State Building Code: User's Guide to 780 CMR Sixth Edition. Commonwealth of Massachusetts.
9. American Institute of Steel Construction. Manual of Steel Construction—Load & Resistance Factor Design. Second Edition. 1995.
10. High Performance Structures Team, The New Civil and Environmental Engineering Building: A Conceptual Design

7.2 *Software used*

AutoCAD R13

Auto Vision R2

SAP2000 Educational

SAP2000 Nonlinear

Matlab Release 5.1

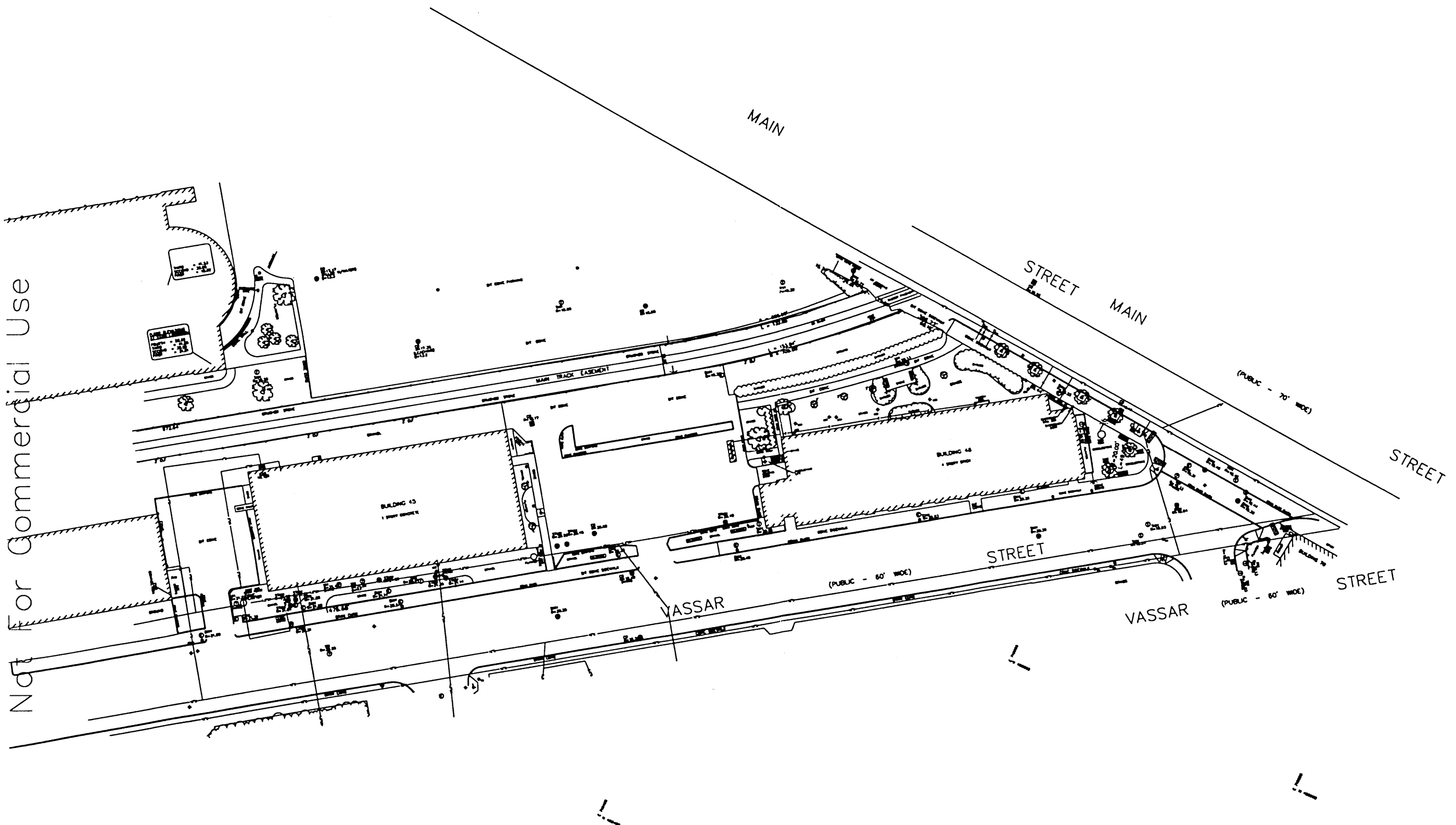
Appendices

A. *Site Maps*

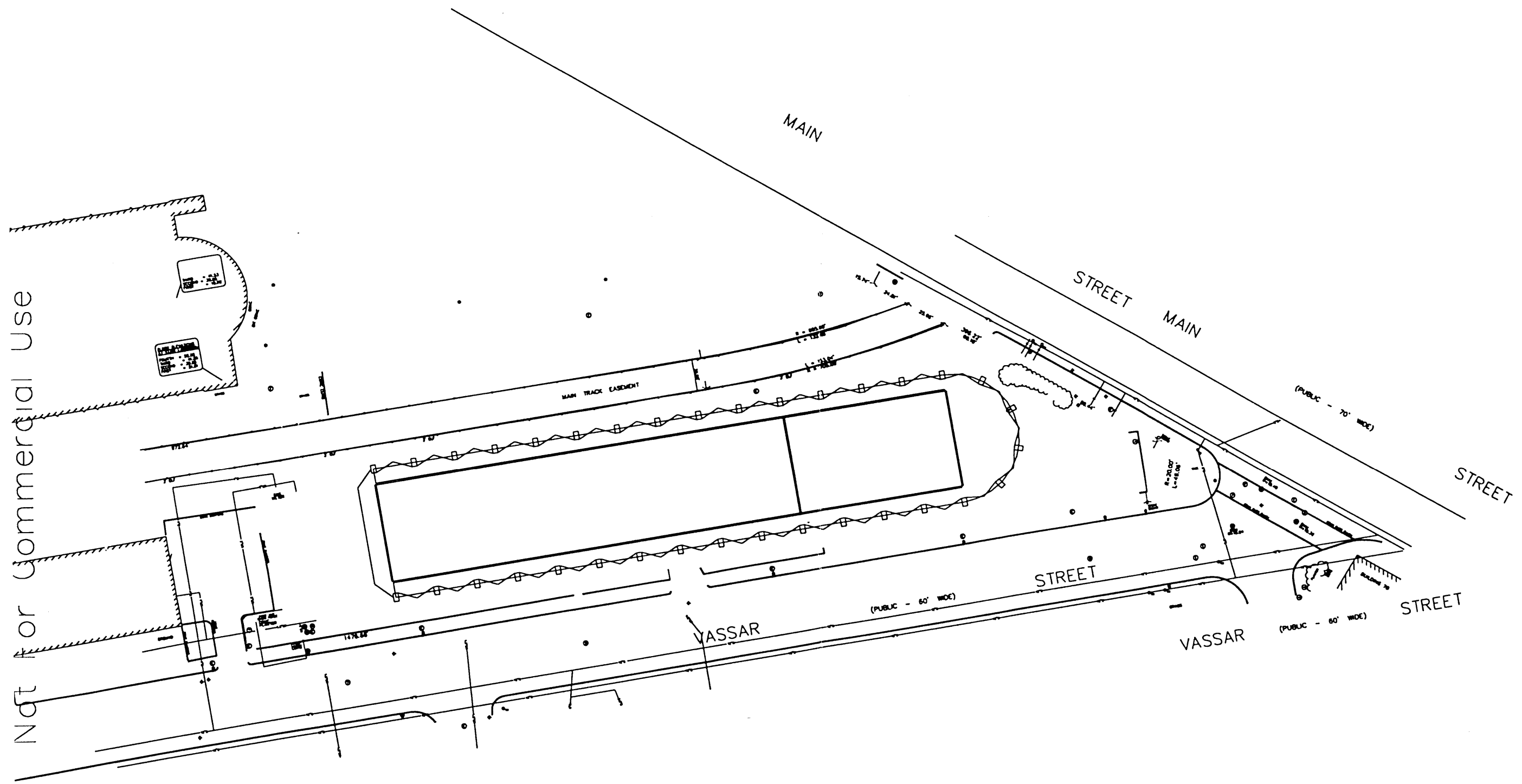
Existing site map Page 39

Proposed building location Page 41

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Not for Commercial Use

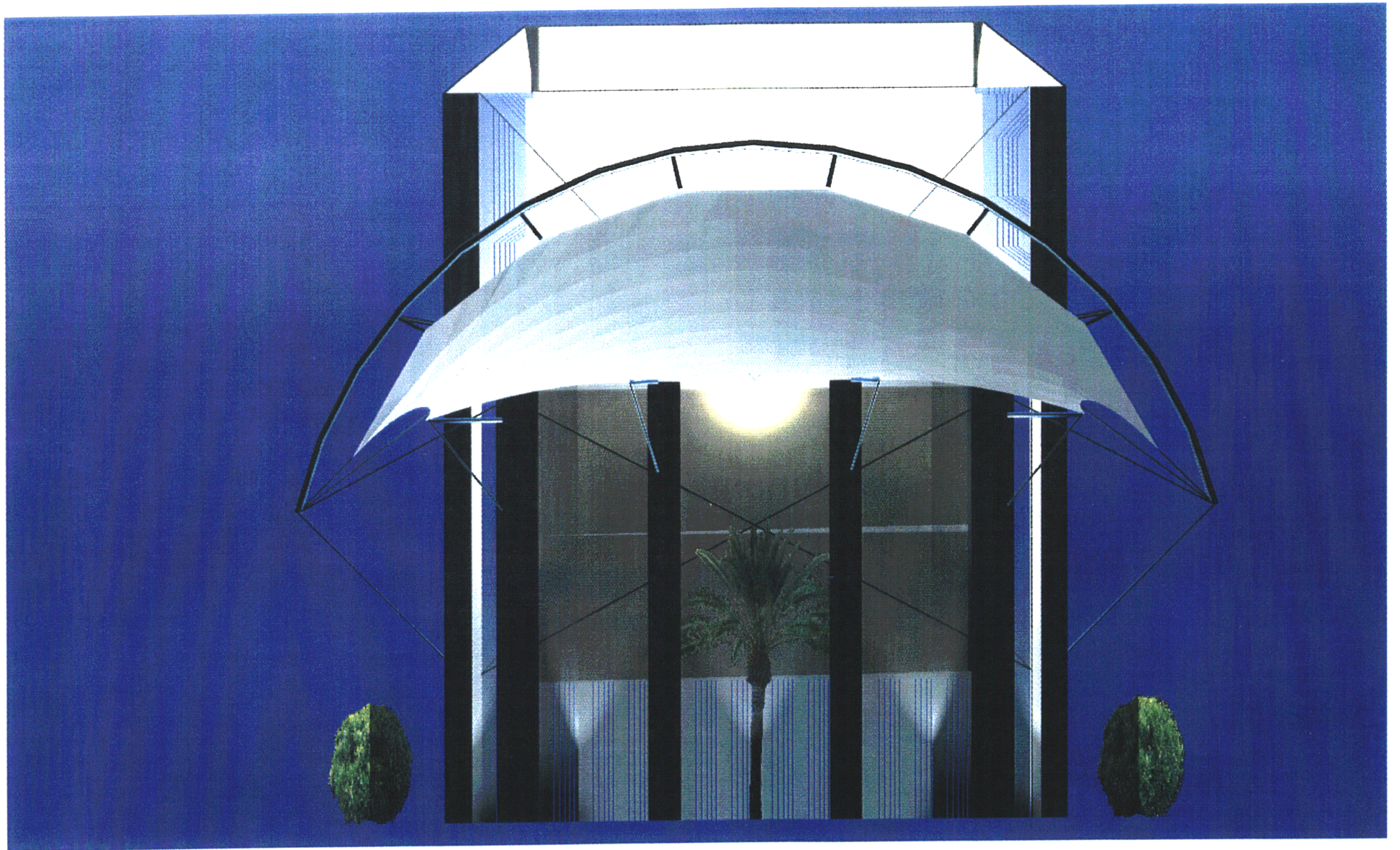


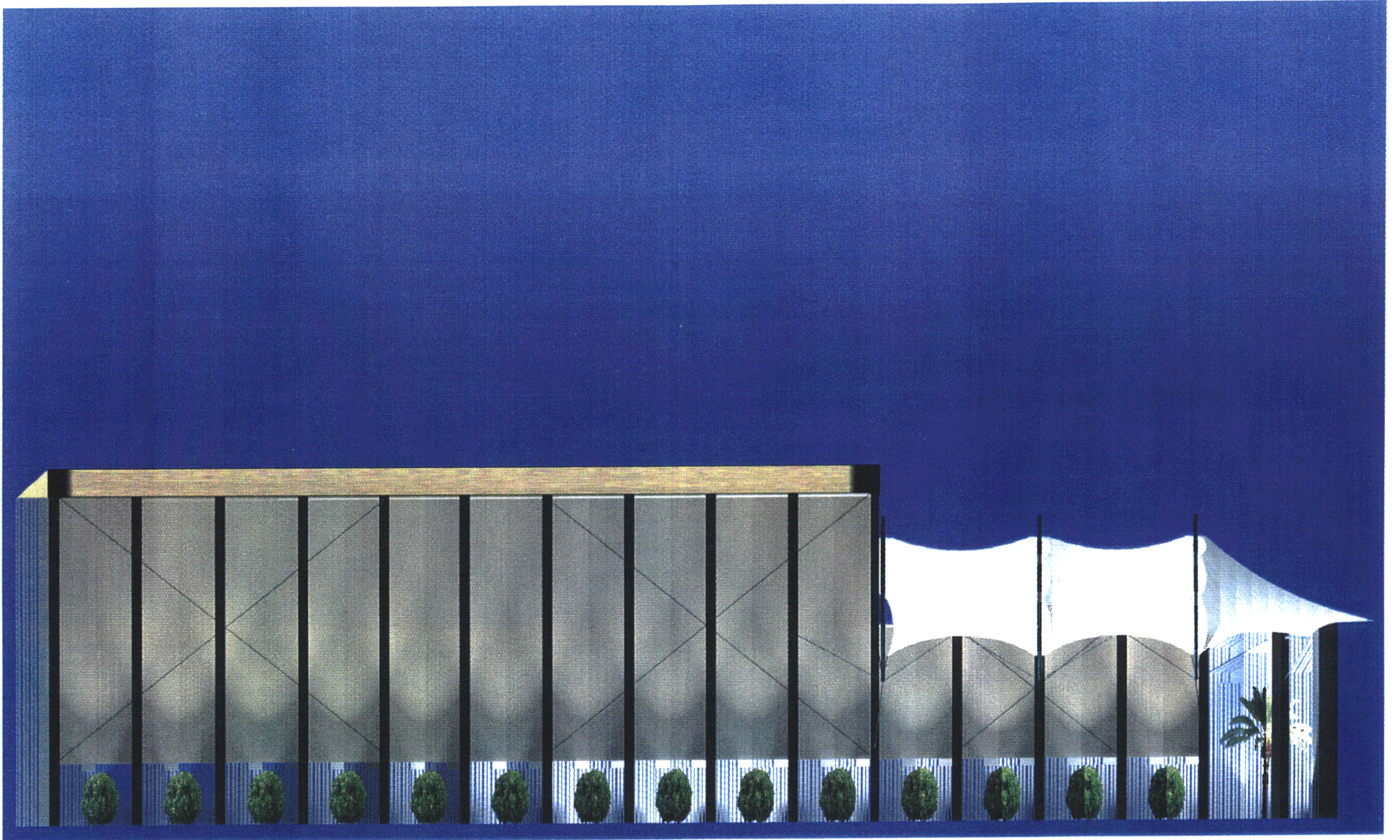
B. Architectural Renderings

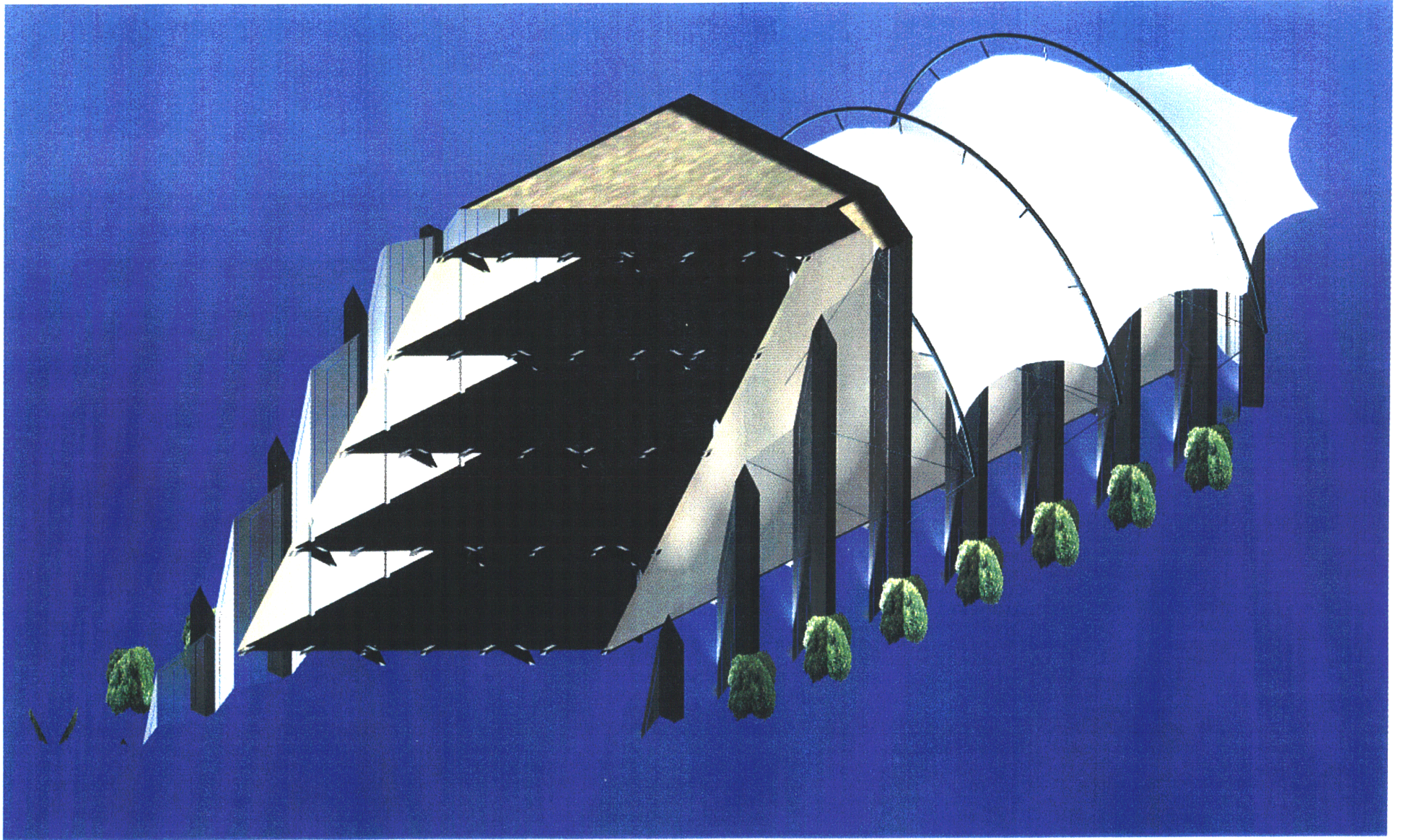
East Elevation Page 45

South Elevation Page 47

Southwest Isometric Cutaway Page 49



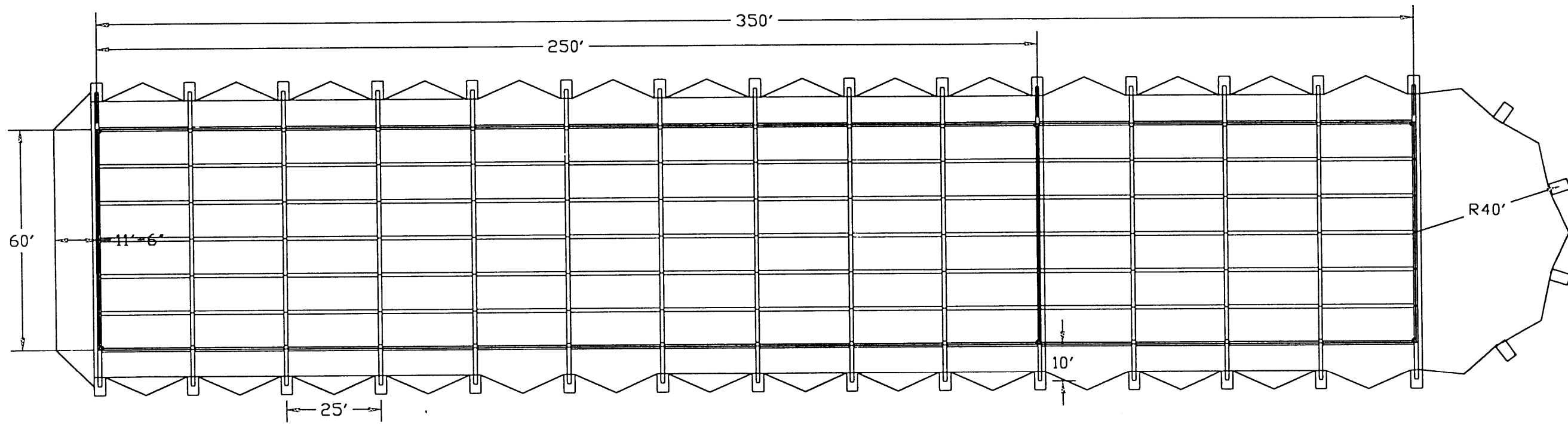




C. Architectural Drawings

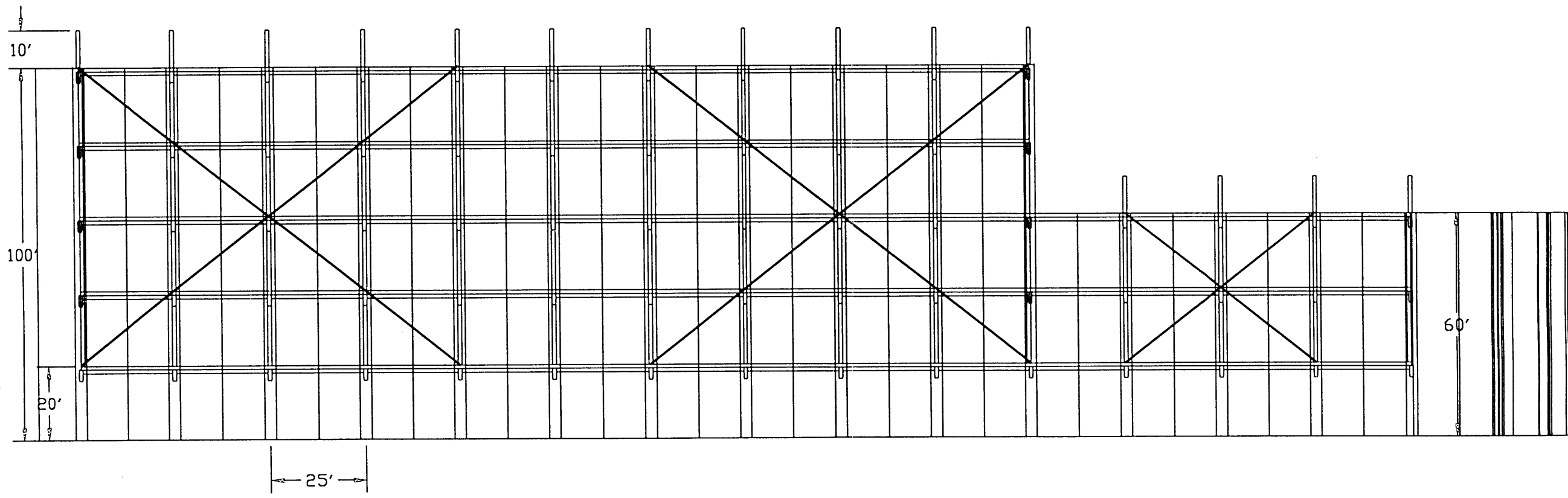
Architectural Drawing A-1	Plan view	Page 53
Architectural Drawing A-2	South elevation	Page 55
Architectural Drawing A-3	South-east isometric view	Page 57

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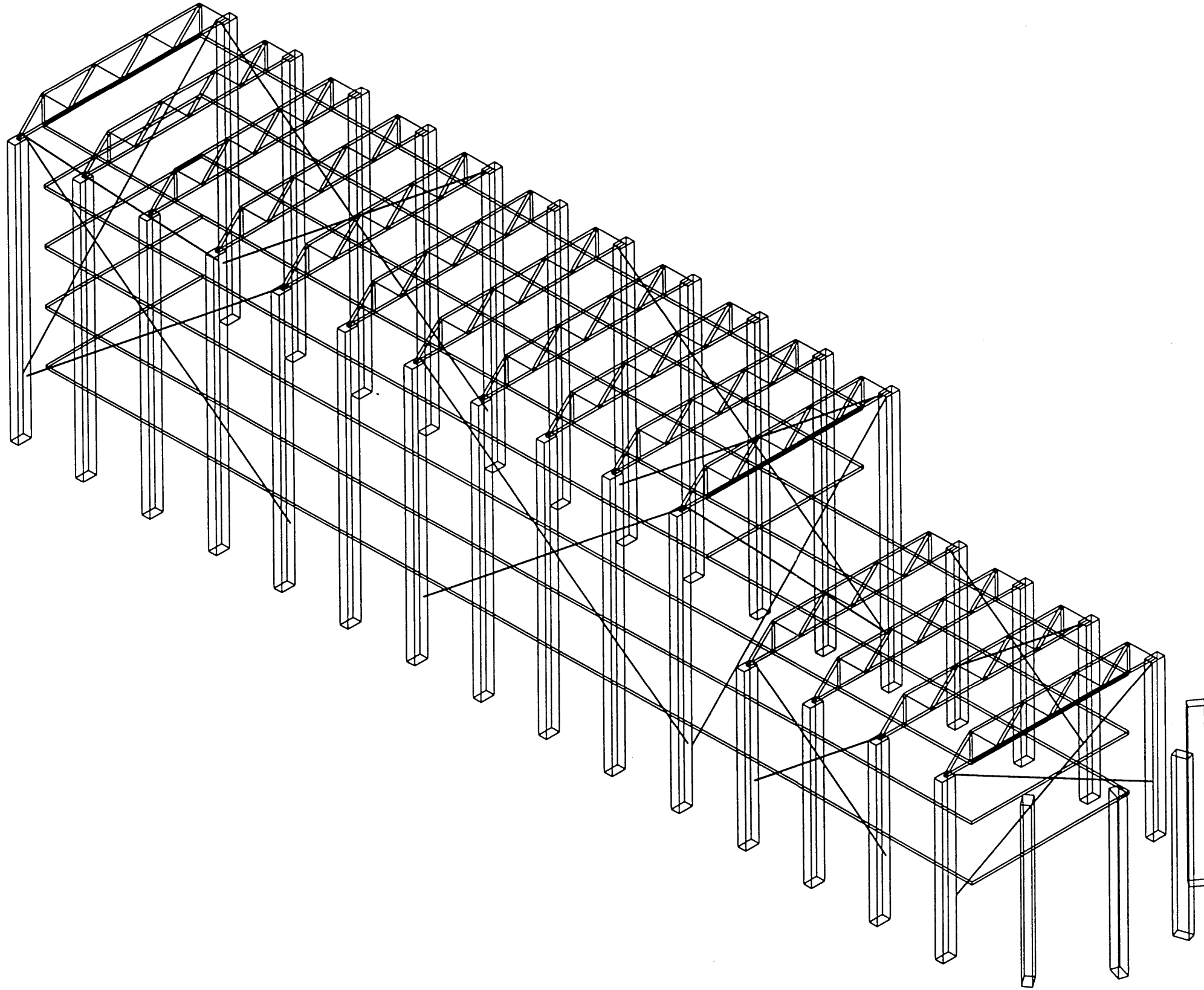
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	Conceptual Design for the New CEE Building	
Group A		Scheme A
		Page: A-1

NOT FOR COMMERCIAL USE



Scale: 1"=30'	Paper Size: 11" x 17"
	Conceptual Design for the New CEE Building
Group A	Scheme A
	Page: A-2

NOT FOR COMMERCIAL USE

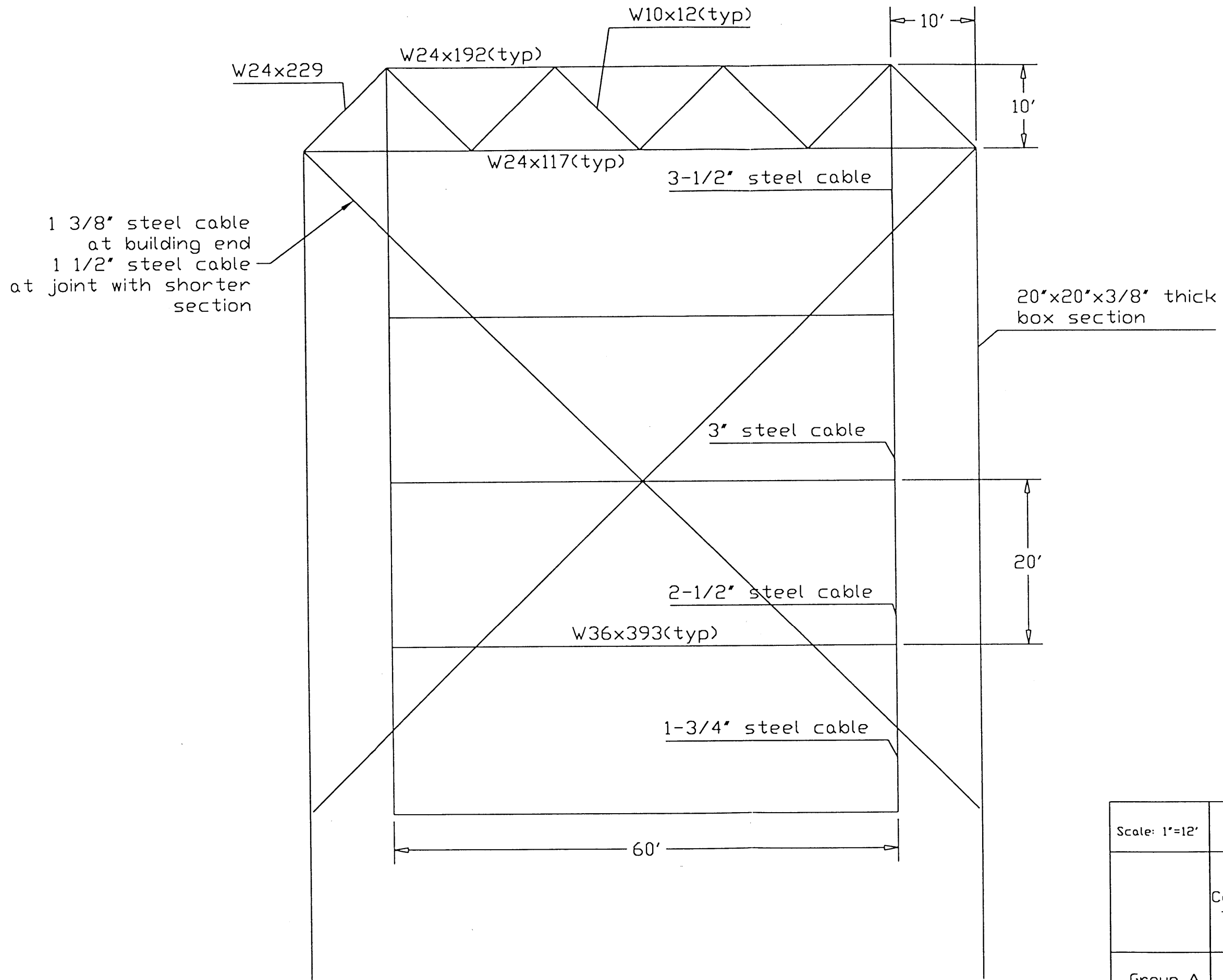


NOT TO SCALE	Paper Size: 11" x 17"	
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Group A		Scheme A
		Page: A-3

D. Structural Drawings

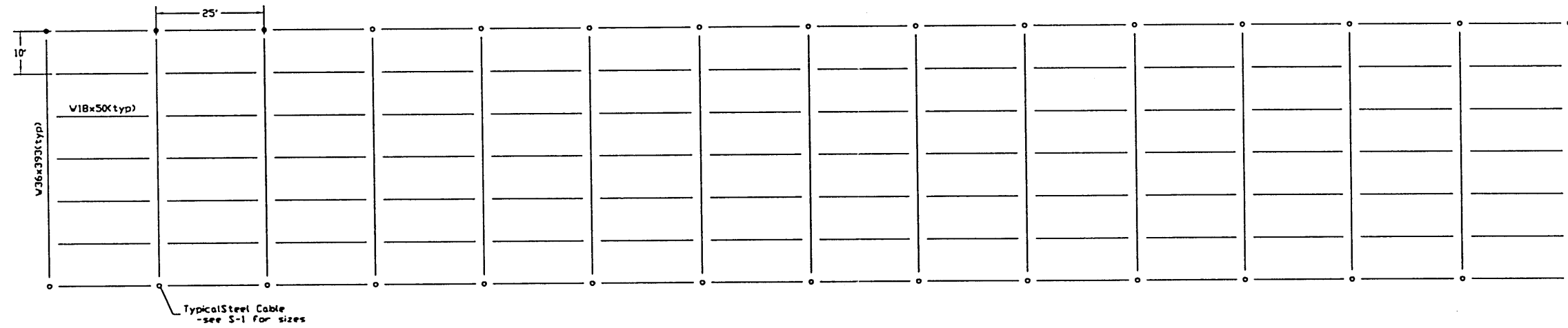
Structural Drawing S-1	Tall portion cross section	Page 61
Structural Drawing S-2	Floor member layout	Page 63
Structural Drawing S-3	Short portion cross section	Page 65
Structural Drawing S-4	South elevation	Page 67

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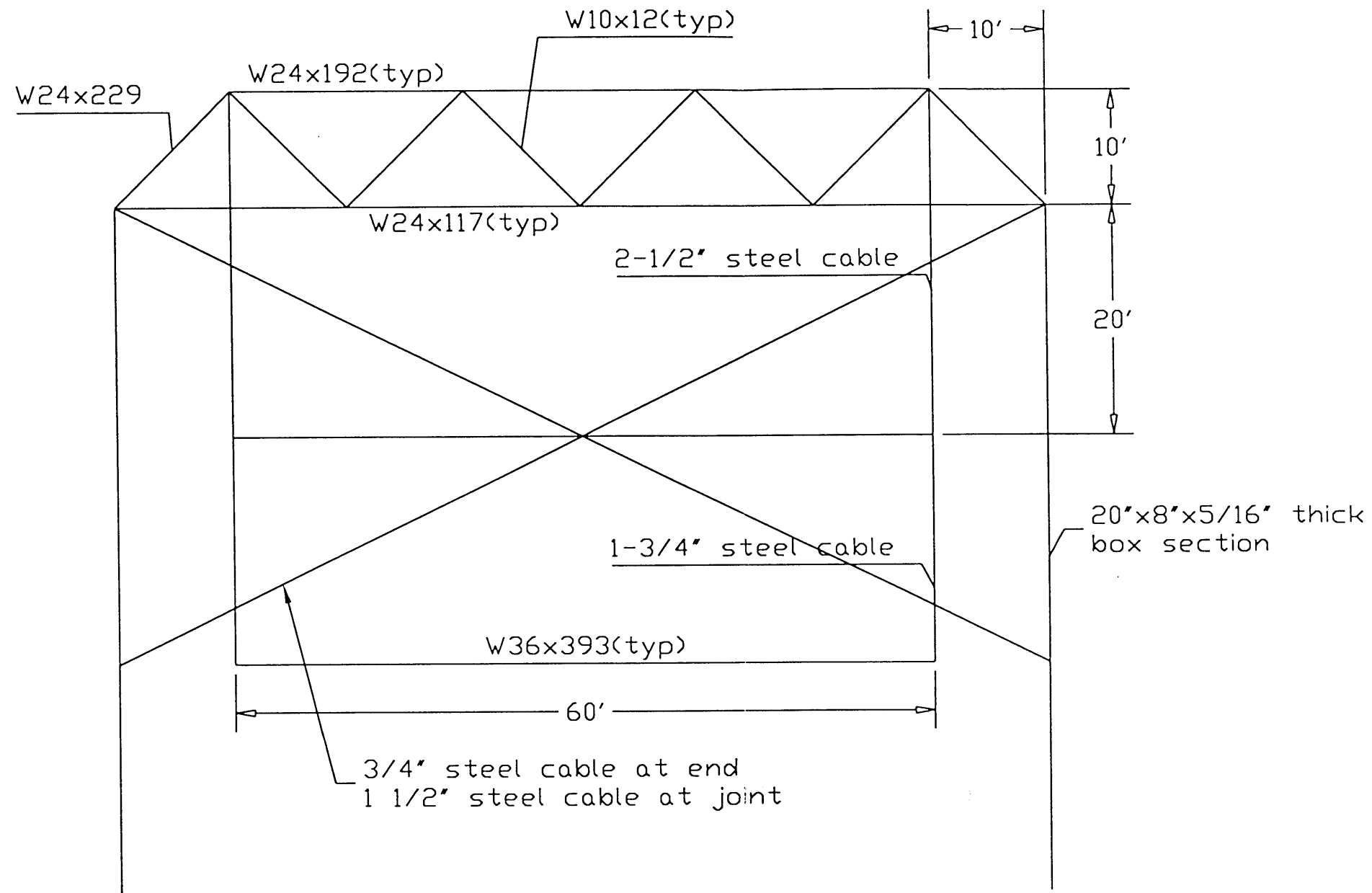
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Group A	Scheme A Page: S-1

Not For Commercial Use



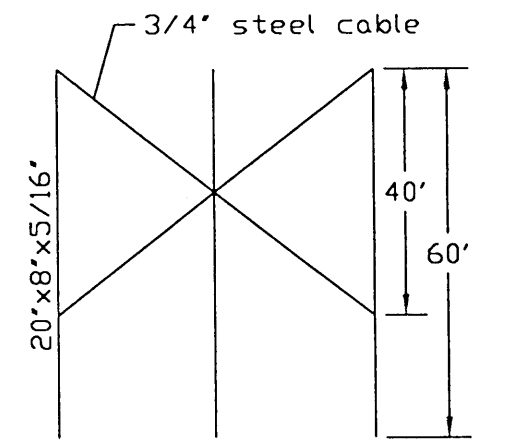
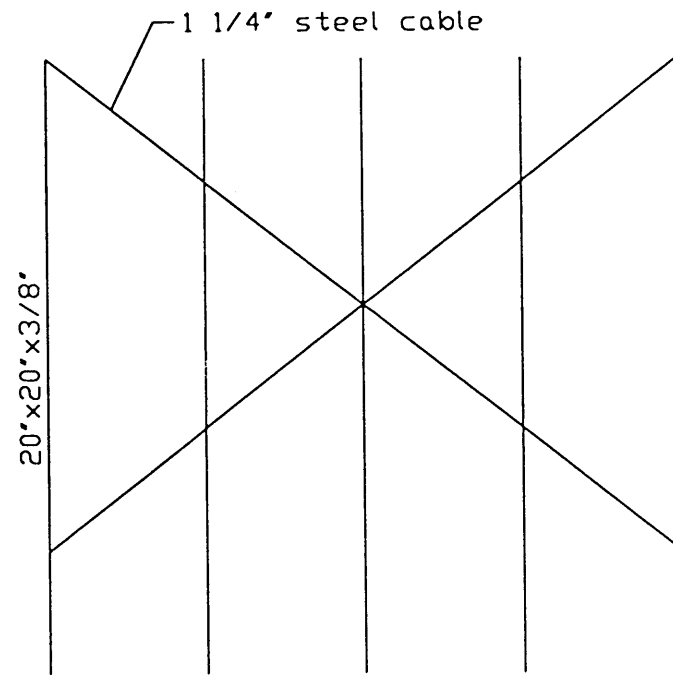
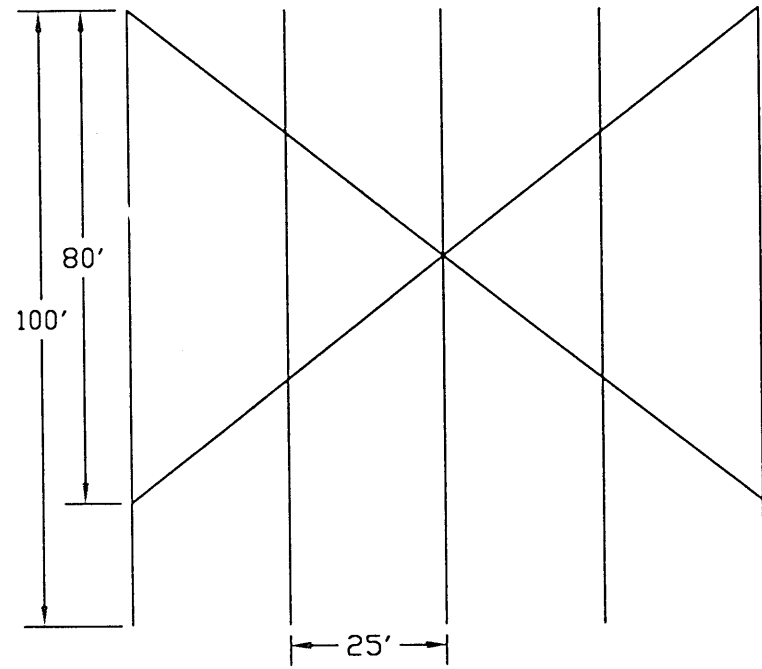
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Group A		Scheme A
		Page: S-2

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Scale: 1"=12'	Paper Size: 11" x 17"
	Conceptual Design for the New CEE Building
Group A	Scheme A
	Page: S-3

NOT FOR COMMERCIAL USE



Scale: 1"-25'	Paper Size: 11" x 17"	
	Conceptual Design for the New CEE Building	
Group A		Scheme A
		Page: S-4

E. Gravity Cable Design

Dead Load = 70 lbs/ft²

Live Load = 160 lbs/ft² (200 lbs/ft² reduced .8 in accordance with mass code)

Factored Load: 1.2(Dead) + 1.6(Live) = 1.2(70 lbs/ft²) + 1.6(160 lbs/ft²) = 340 lbs/ft²

Column load per floor based on tributary area: (25 ft)(30 ft)(340 lbs/ft²) = 255 kips

Assume 120ksi steel cables

2nd Story

$$\frac{255kips}{120ksi} = 2.125in^2$$

$$D = 2\sqrt{\frac{2.125}{\pi}} = 1.65in \quad \text{use } 1 \frac{3}{4}''$$

3rd Story

$$\frac{2(255kips)}{120ksi} = 4.25in^2$$

$$D = 2\sqrt{\frac{4.25}{\pi}} = 2.33in \quad \text{use } 2 \frac{1}{2}''$$

4th Story

$$\frac{3(255kips)}{120ksi} = 6.375in^2$$

$$D = 2\sqrt{\frac{6.375}{\pi}} = 2.85in \quad \text{use } 3''$$

5th Story

$$\frac{4(255kips)}{120ksi} = 8.5in^2$$

$$D = 2\sqrt{\frac{8.5}{\pi}} = 3.29in \quad \text{use } 3 \frac{1}{2}'' \text{ Calculations}$$

F. Exterior Column Design

$$L = 100\text{ft}$$

$$F_y = 50 \text{ ksi}$$

$$K = 2.0 \text{ (fixed at base free at top)}$$

Tall Columns

Max Axial load (including self weight) = 1170 kips (from SAP 2000 Output)

try 20" x 20" $^{-3/8}$ " thick

$$\frac{KL}{R} = \frac{(2.0)(100\text{ft})}{7.99\text{in}} = 25.0 \quad \text{from Table E-1 (LRFD vol.1)} \phi_c F_{cr} = 40.60 \text{ ksi}$$

$$\frac{\text{Load}}{\phi_c F_{cr}} = \frac{1170\text{kips}}{40.60\text{ksi}} = 28.8\text{in}^2 \quad \text{area of section} = 29.1\text{in}^2 \quad \text{O.K.}$$

Short Columns

Max Axial load (including self weight) = 558 kips (from SAP 2000 Output)

try 20" x 8" $^{-5/16}$ " thick

$$\frac{KL}{R} = \frac{(2.0)(60\text{ft})}{3.46\text{in}} = 35.50 \quad \text{from Table E-1 (LRFD vol.1)} \phi_c F_{cr} = 38.66 \text{ ksi}$$

$$\frac{\text{Load}}{\phi_c F_{cr}} = \frac{558\text{kips}}{38.86\text{ksi}} = 14.4\text{in}^2 \quad \text{area of section} = 16.9 \text{ in}^2 \quad \text{O.K.}$$

G. Lateral Bracing Cable Design

Assume 120 ksi cables

Cables must be fireproofed short way

Cables the long way are far enough from the main building in case of fire

Dead: beams	5 psf	Snow: 30 psf
girders	16 psf	
slab	34 psf	
partitions	10 psf	
mechanical	<u>5 psf</u>	
TOTAL:	70 psf	

$$C_s = \frac{1.2A_v S}{RT^{\frac{2}{3}}}$$

$$A_v = .12g$$

$$S = S_4 = 2.0$$

$$C_s = \frac{1.2(.12)(2.0)}{(7)(1.0)^{\frac{2}{3}}} = .0411$$

R = 7 eccentrically braced non-moment frame

$$T = 1.0 \text{ sec}$$

Tall Section Bracing

$$\text{Floor Area: } (250 \text{ ft})(60 \text{ ft}) = 15,000 \text{ ft}^2 \quad 5 \text{ floors} = 75,000 \text{ ft}^2$$

$$\text{Floor Weight: } (75,000 \text{ ft}^2)(70 \text{ psf}) = 5250 \text{ kips}$$

$$\text{Roof Area: } 15,000 \text{ ft}^2$$

$$\text{Roof Weight: } (15,000 \text{ ft}^2)(30 \text{ psf}) = 450 \text{ kips}$$

$$\text{Seismic Design Weight: } 1.0(\text{Dead}) + .5(\text{Snow}) = 5250 \text{ kips} + .5(450 \text{ kips}) = 5475 \text{ kips}$$

$$V_{\text{tall}} = C_s W = (.0411)(5465 \text{ kips}) = 225.3 \text{ kips} \approx 226 \text{ kips}$$

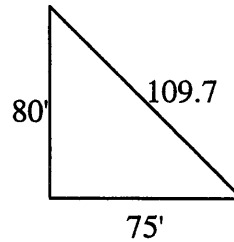
Across Building

$$\frac{226 \text{ kips}}{2 \text{ braces}} = 113 \text{ kips / brace}$$

$$113 \text{ kips} \left(\frac{109.7 \text{ ft}}{75 \text{ ft}} \right) = 165.2 \text{ kips}$$

$$\frac{165.2 \text{ kips}}{120 \text{ ksi}} = 1.38 \text{ in}^2$$

$$D = 2 \sqrt{\frac{1.38}{\pi}} = 1.32 \text{ in} \quad \text{use } 1 \frac{3}{8} \text{''}$$



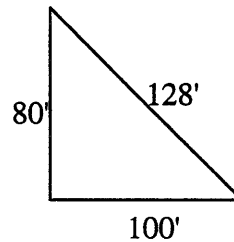
Along Building

$$\frac{226 \text{ kips}}{2 \text{ braces}} = 113 \text{ kips / brace}$$

$$113 \text{ kips} \left(\frac{128 \text{ ft}}{100 \text{ ft}} \right) = 144.7 \text{ kips}$$

$$\frac{144.7 \text{ kips}}{120 \text{ ksi}} = 1.21 \text{ in}^2$$

$$D = 2 \sqrt{\frac{1.21}{\pi}} = 1.24 \text{ in} \quad \text{use } 1 \frac{1}{4} \text{''}$$



Short Section Bracing

$$\text{Floor Area: } (1000 \text{ ft})(60 \text{ ft}) = 6,000 \text{ ft}^2 \quad 3 \text{ floors} = 18,000 \text{ ft}^2$$

$$\text{Floor Weight: } (18,000 \text{ ft}^2)(70 \text{ psf}) = 1260 \text{ kips}$$

$$\text{Roof Area: } 6,000 \text{ ft}^2$$

$$\text{Roof Weight: } (6,000 \text{ ft}^2)(30 \text{ psf}) = 180 \text{ kips}$$

$$\text{Seismic Design Weight: } 1.0(\text{Dead}) + .5(\text{Snow}) = 1260 \text{ kips} + .5(180 \text{ kips}) = 1350 \text{ kips}$$

$$V_{\text{tall}} = C_s W = (.0411)(1350 \text{ kips}) = 55.5 \text{ kips} \approx 56 \text{ kips}$$

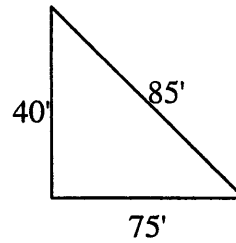
Across Building

$$\frac{56 \text{ kips}}{2 \text{ braces}} = 28 \text{ kips / brace}$$

$$28 \text{ kips} \left(\frac{85 \text{ ft}}{75 \text{ ft}} \right) = 31.7 \text{ kips}$$

$$\frac{31.7 \text{ kips}}{120 \text{ ksi}} = .264 \text{ in}^2$$

$$D = 2 \sqrt{\frac{.264}{\pi}} = .58 \text{ in} \quad \text{use } \frac{3}{4} \text{''}$$



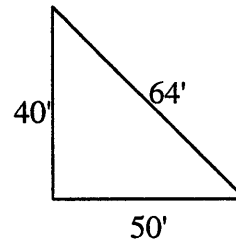
Along Building

$$\frac{56 \text{ kips}}{2 \text{ braces}} = 28 \text{ kips / brace}$$

$$28 \text{ kips} \left(\frac{64 \text{ ft}}{50 \text{ ft}} \right) = 35.84 \text{ kips}$$

$$\frac{35.84 \text{ kips}}{120 \text{ ksi}} = .30 \text{ in}^2$$

$$D = 2 \sqrt{\frac{.30}{\pi}} = .62 \text{ in} \quad \text{use } \frac{3}{4} \text{''}$$



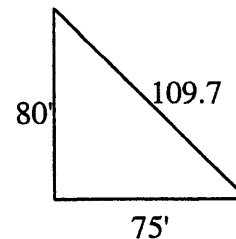
Joint between Tall and Short Section

$$\frac{(226 + 56) \text{ kips}}{2 \text{ braces}} = 141 \text{ kips / brace}$$

$$141 \text{ kips} \left(\frac{109.7 \text{ ft}}{75 \text{ ft}} \right) = 206.3 \text{ kips}$$

$$\frac{206.3 \text{ kips}}{120 \text{ ksi}} = 1.72 \text{ in}^2$$

$$D = 2 \sqrt{\frac{1.72}{\pi}} = 1.48 \text{ in}$$



H. Geotechnical Requirements

Column Loads

Tall Columns: **20" x 20" -³/₈" thick (box sections)**

Axial Load: Design: 1208 kips
 Dead: 310 kips

Shear Load: 113 kips across or along building, depends on location
 183 kips at base of column at joint with short section

Moment: 2260 kip-ft

Short Columns: **20" x 8" -⁵/₁₆" thick (box sections)**

Axial Load: Design: 572 kips
 Dead: 175 kips

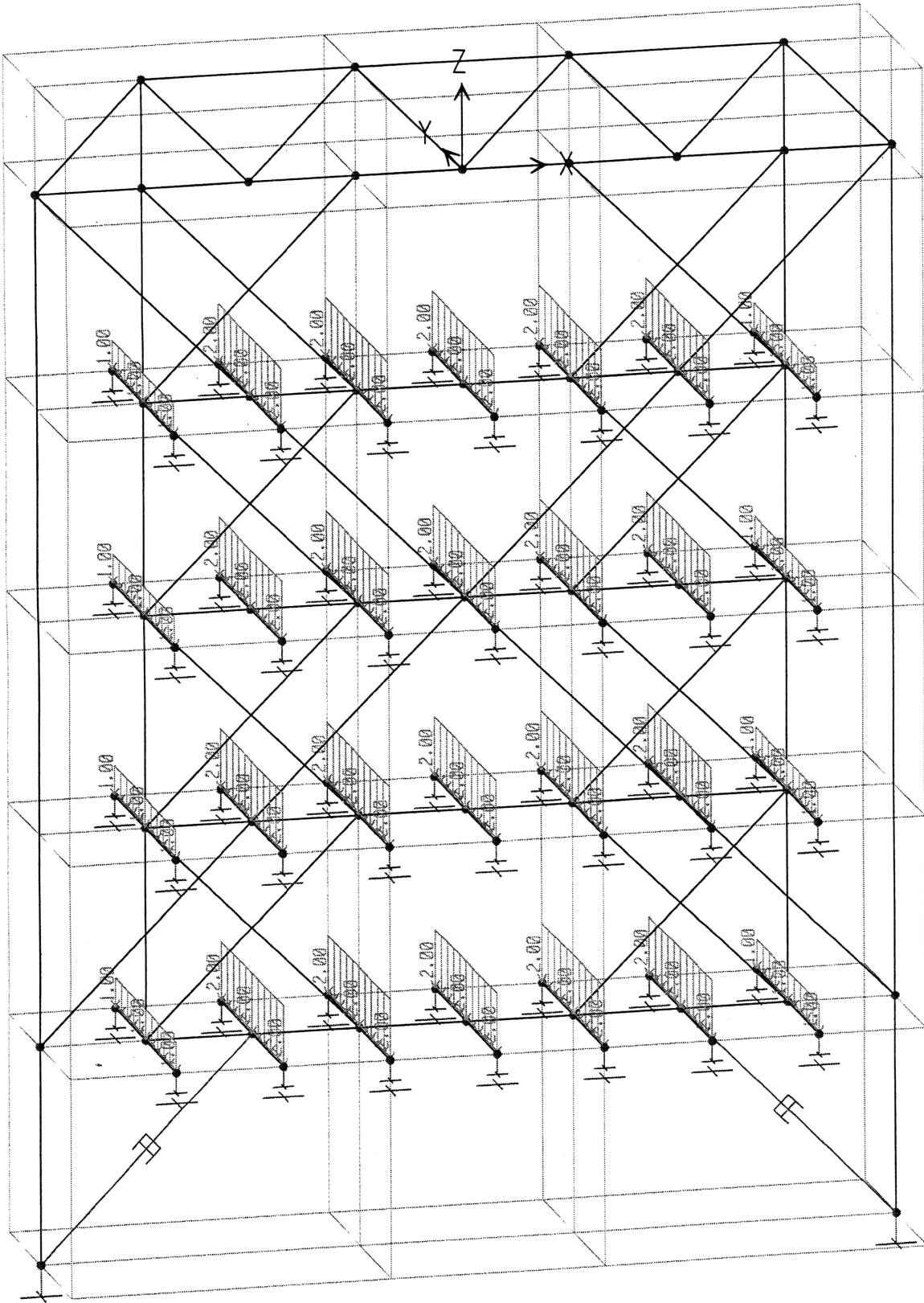
Shear Load: 70 kips

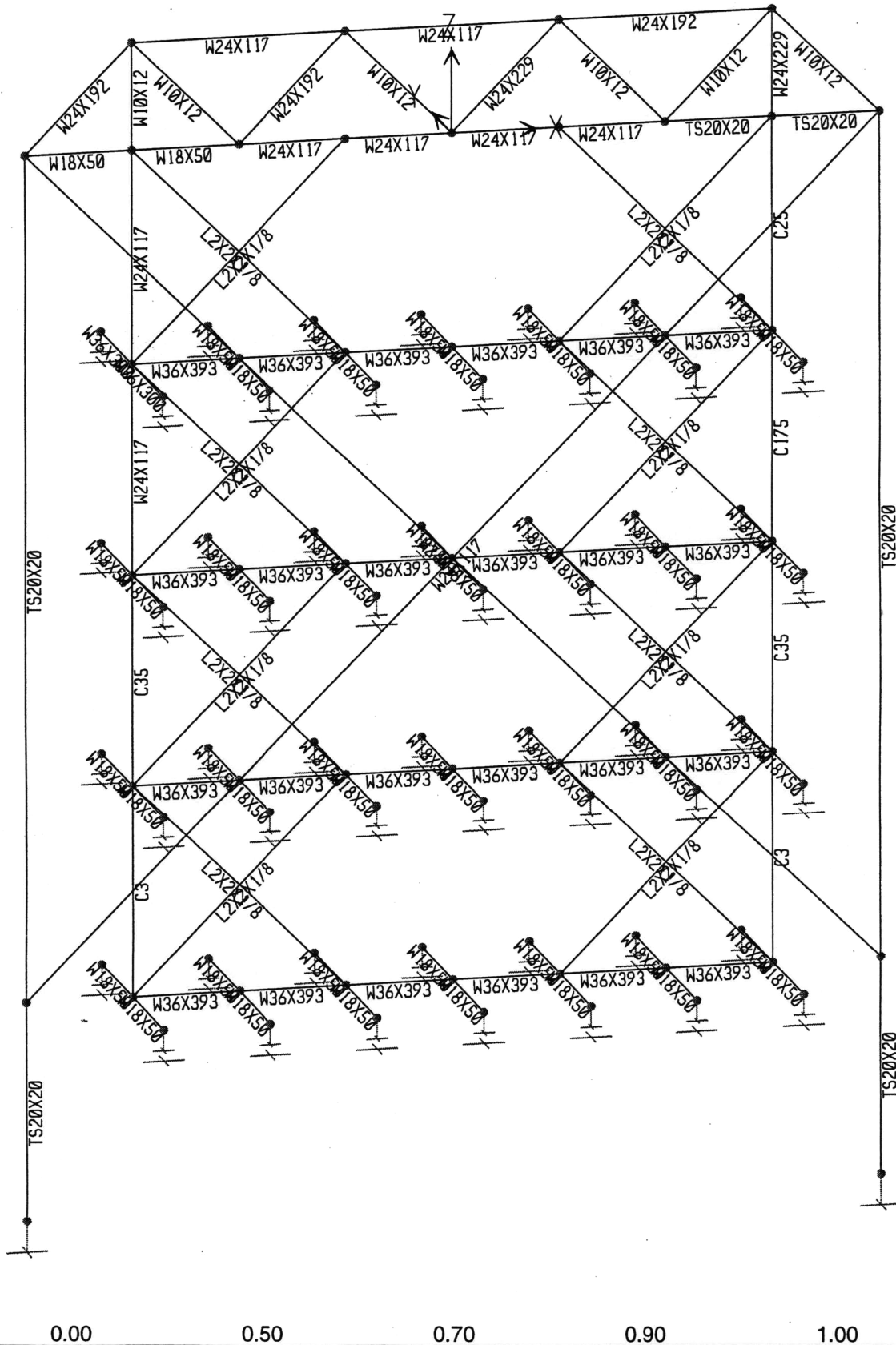
Moment: 1390 kip-ft

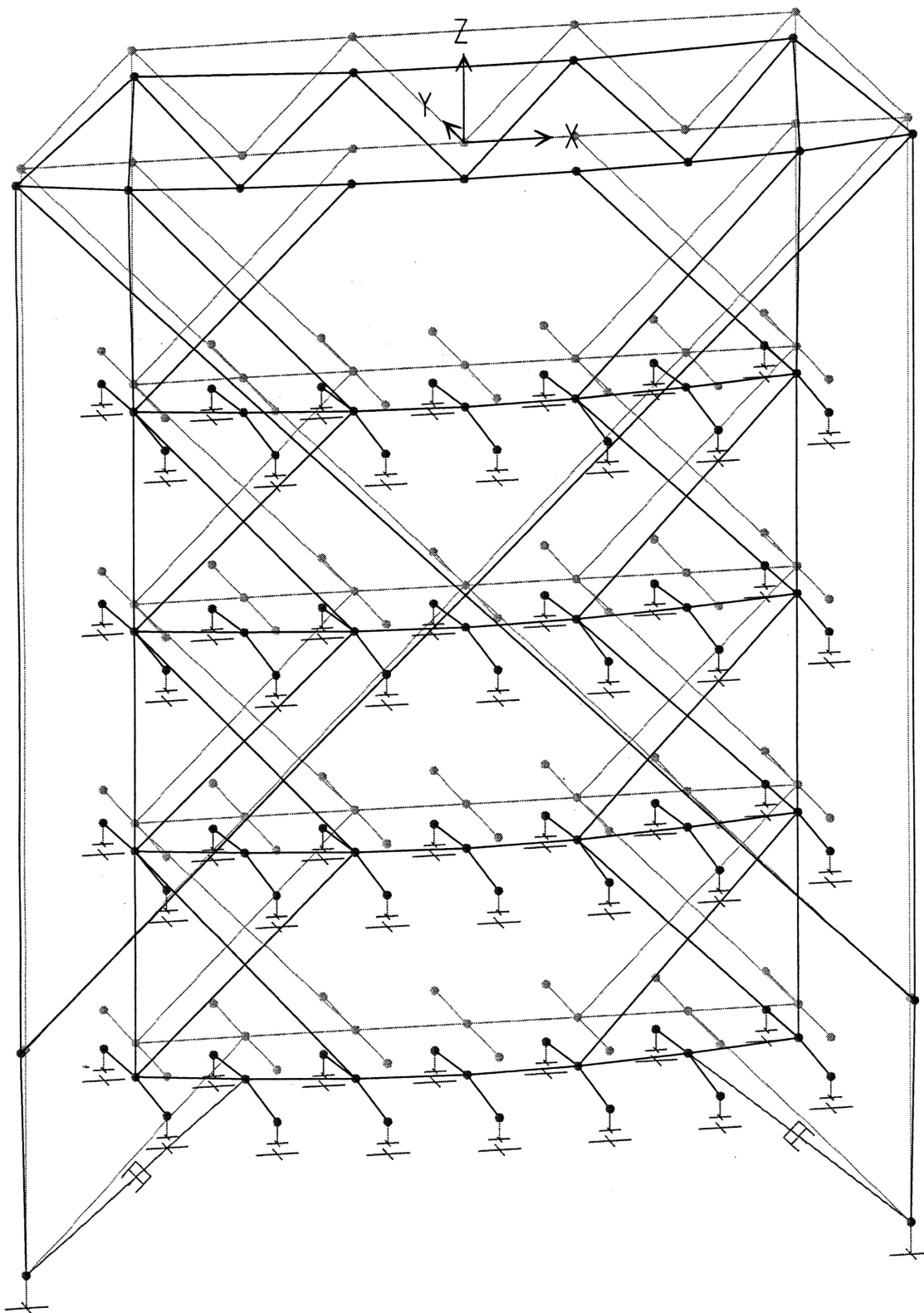
I. SAP 2000 Output

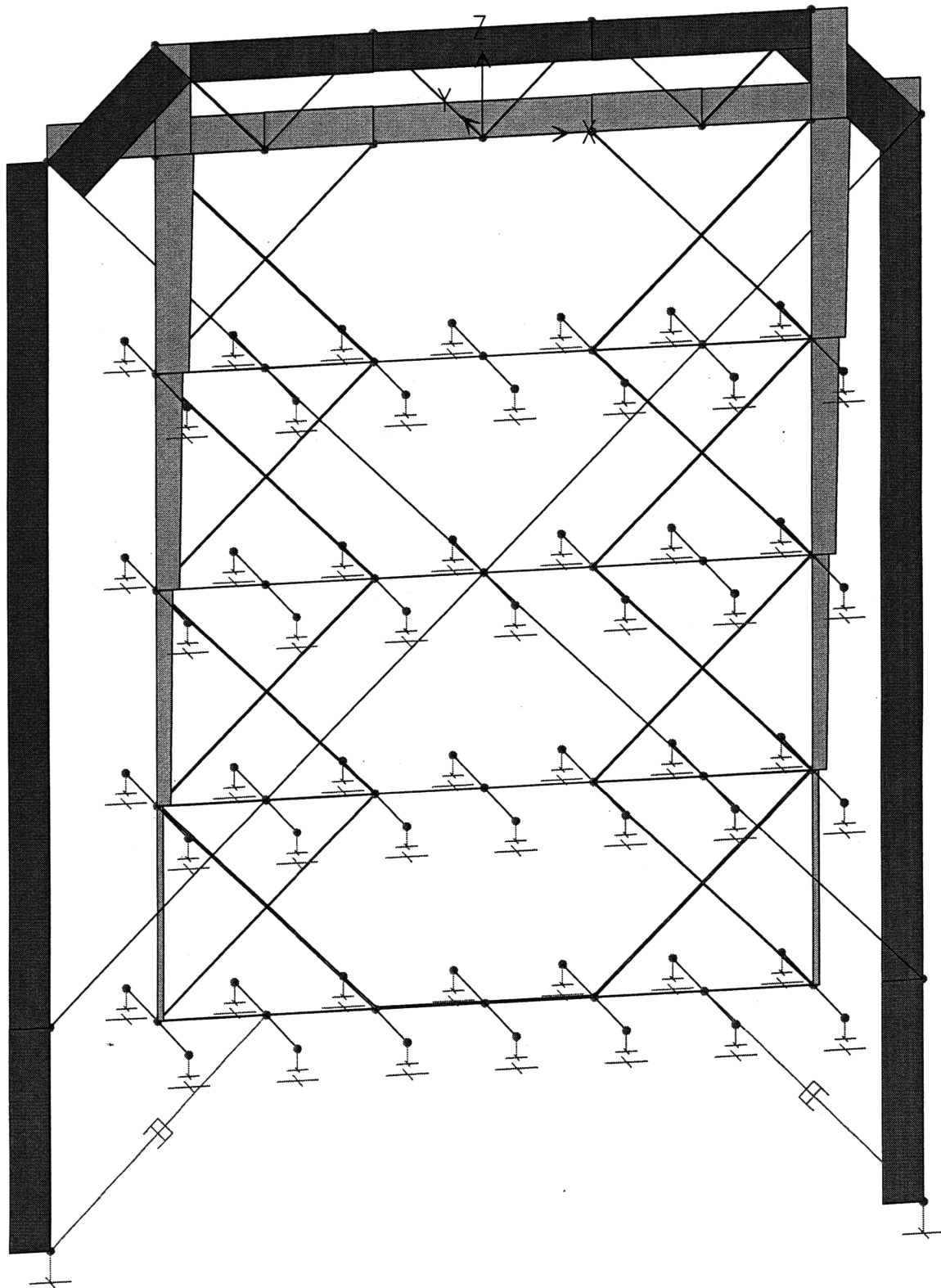
Frame Span Loads	Page 77
Steel Design Sections	Page 78
Deformed Shape Due To Live Load	Page 79
Axial Force Diagram Due To Live Load	Page 80
Shear Force Diagram Due To Live Load	Page 81
Moment Diagram Due To Live Load	Page 82
First Mode Shape	Page 83
Second Mode Shape	Page 84
Fourth Mode Shape	Page 85
Joint 5 Undamped Displacement (El Centro)	Page 86
Joint 18 Undamped Displacement (El Centro)	Page 87
Joint 5 Undamped Displacement (Northridge)	Page 88
Joint 18 Undamped Displacement (Northridge)	Page 89
Steel Design Sections With Dampers	Page 90
Joint 5 30 kip Damped Displacement (El Centro)	Page 91
Joint 18 30 kip Damped Displacement (El Centro)	Page 92
Joint 5 30 kip Damped Displacement (Northridge)	Page 93
Joint 18 30 kip Damped Displacement (Northridge)	Page 94
Joint 5 60 kip Damped Displacement (El Centro)	Page 95
Joint 18 60 kip Damped Displacement (El Centro)	Page 96
Joint 5 60 kip Damped Displacement (Northridge)	Page 97
Joint 18 60 kip Damped Displacement (Northridge)	Page 98

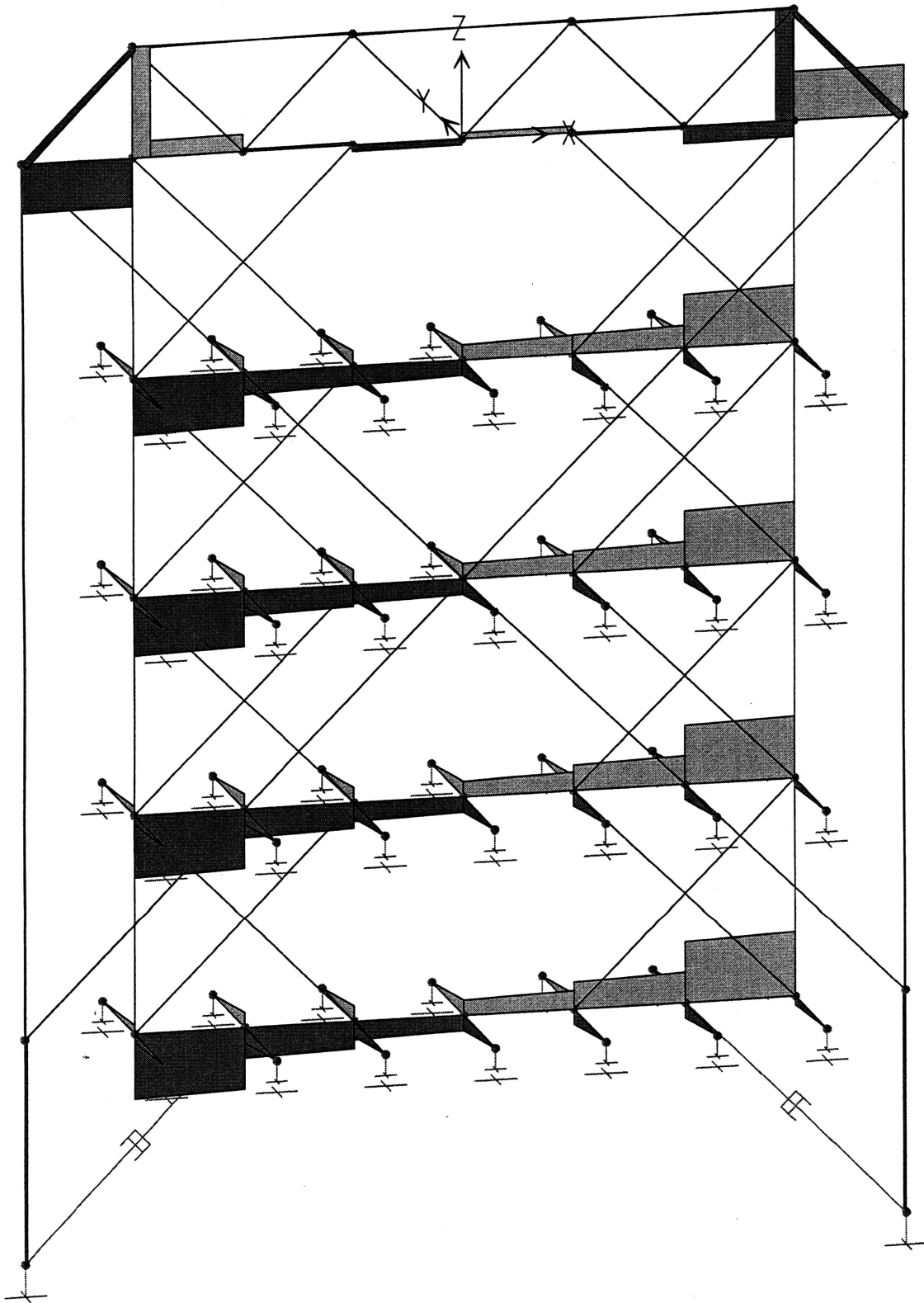
J. Cost Analysis Spreadsheet

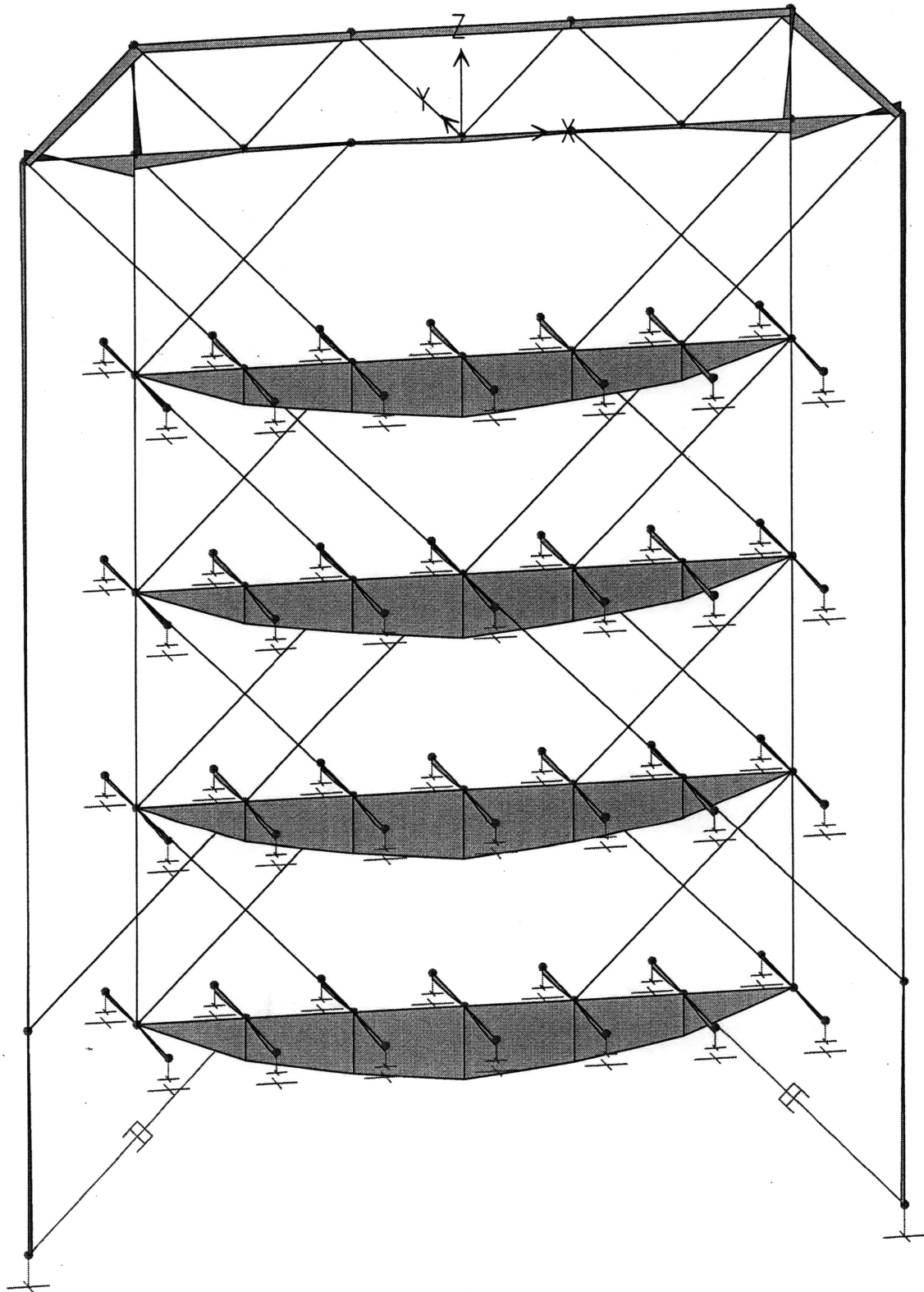


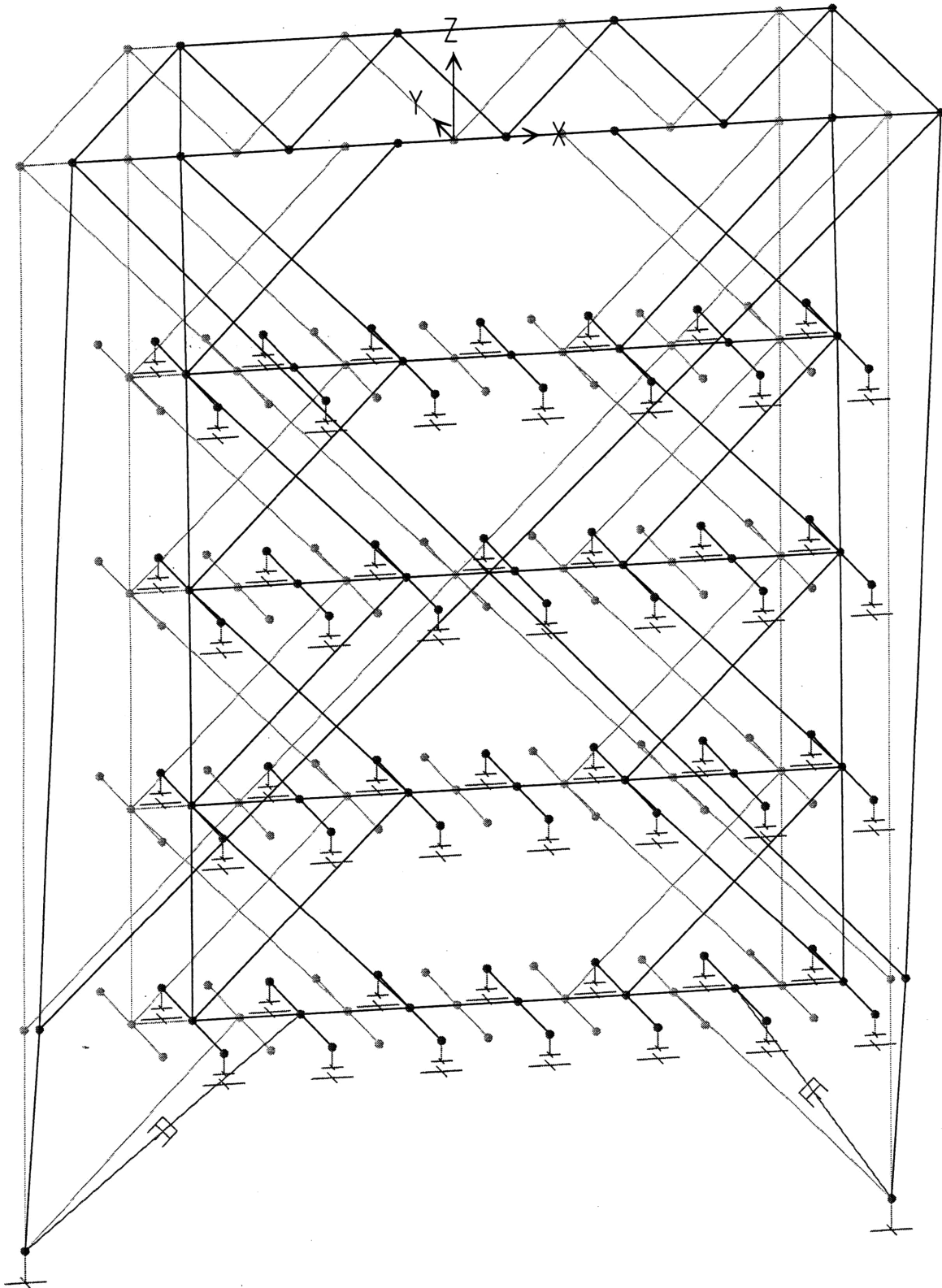


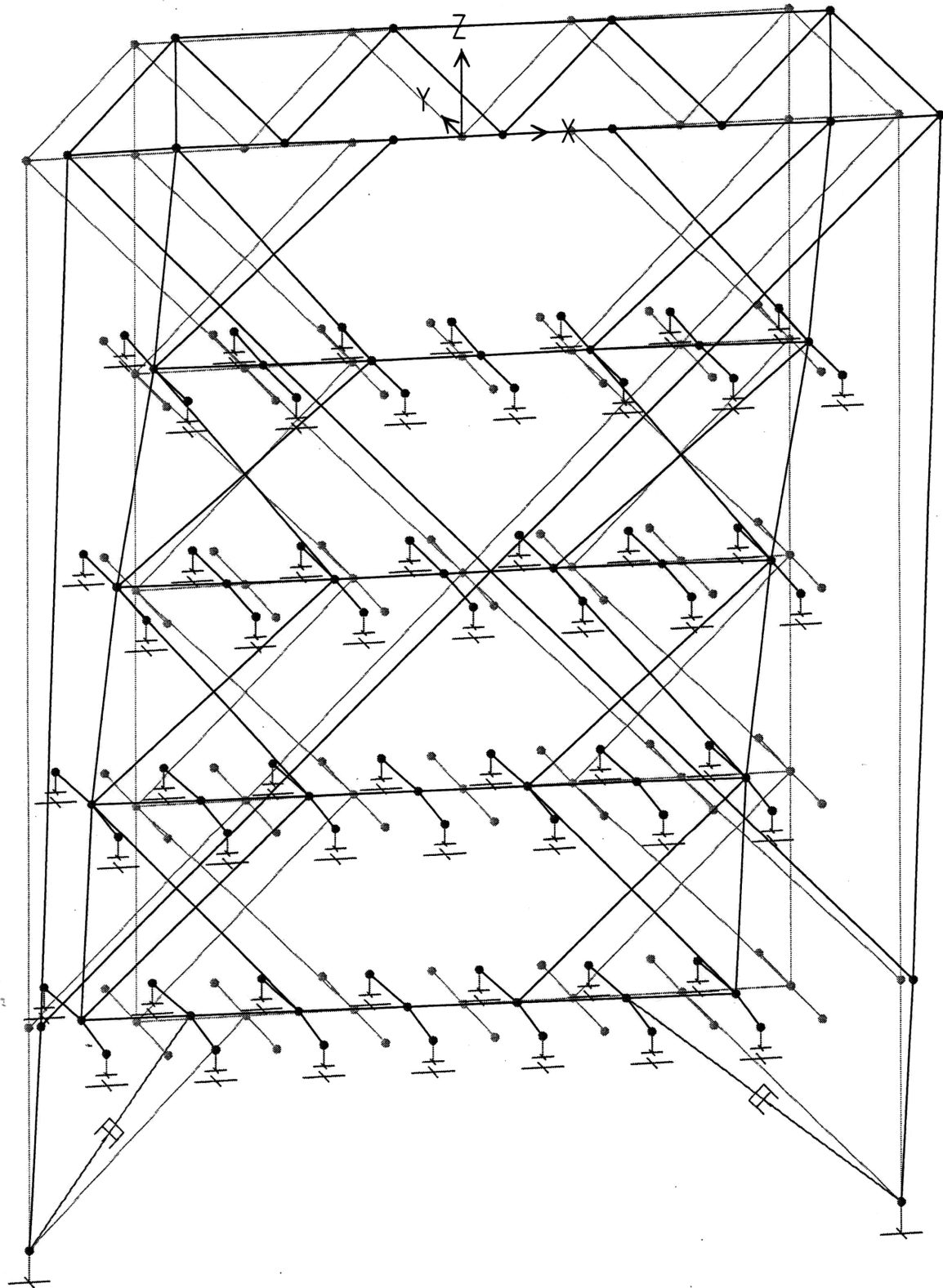


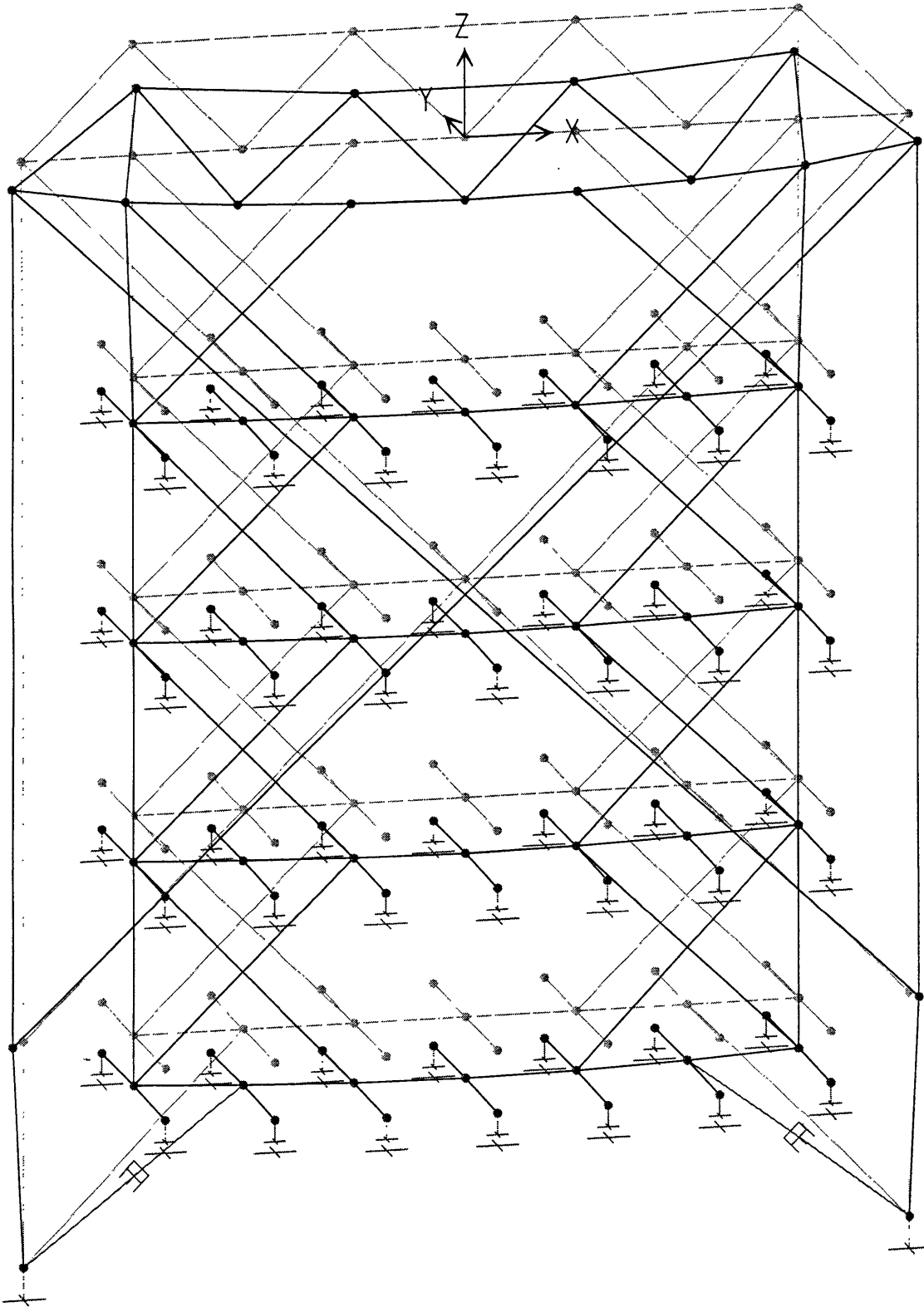


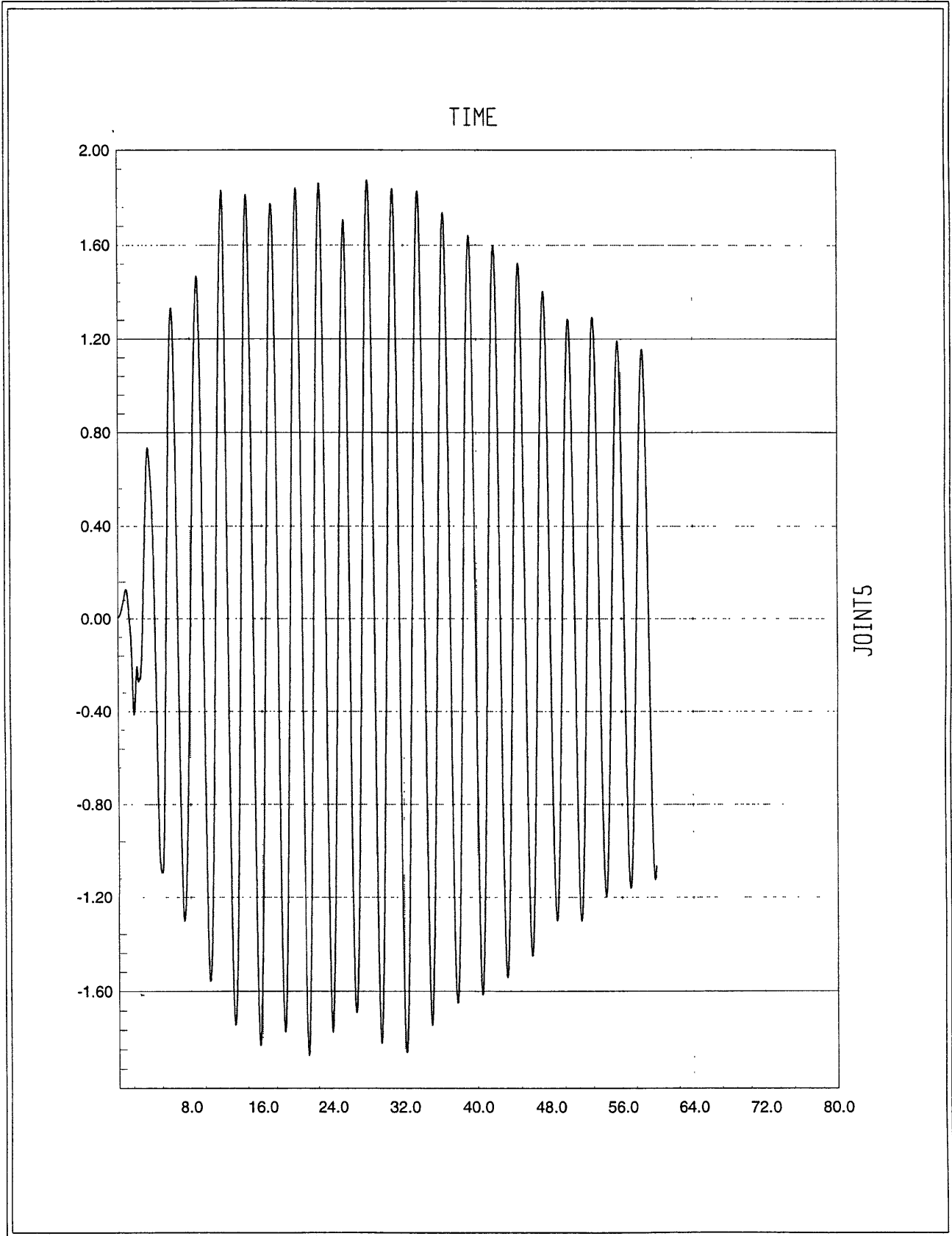




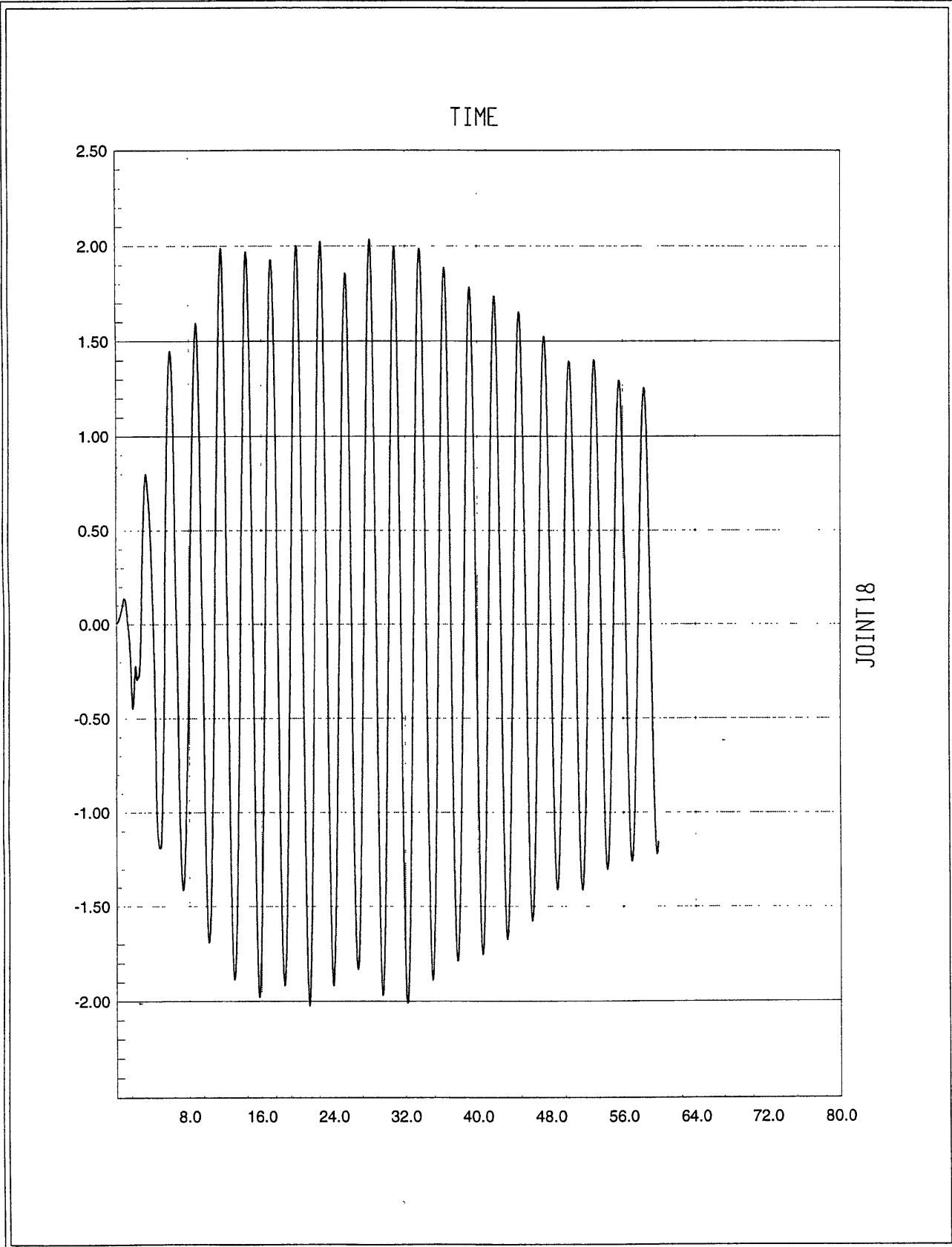




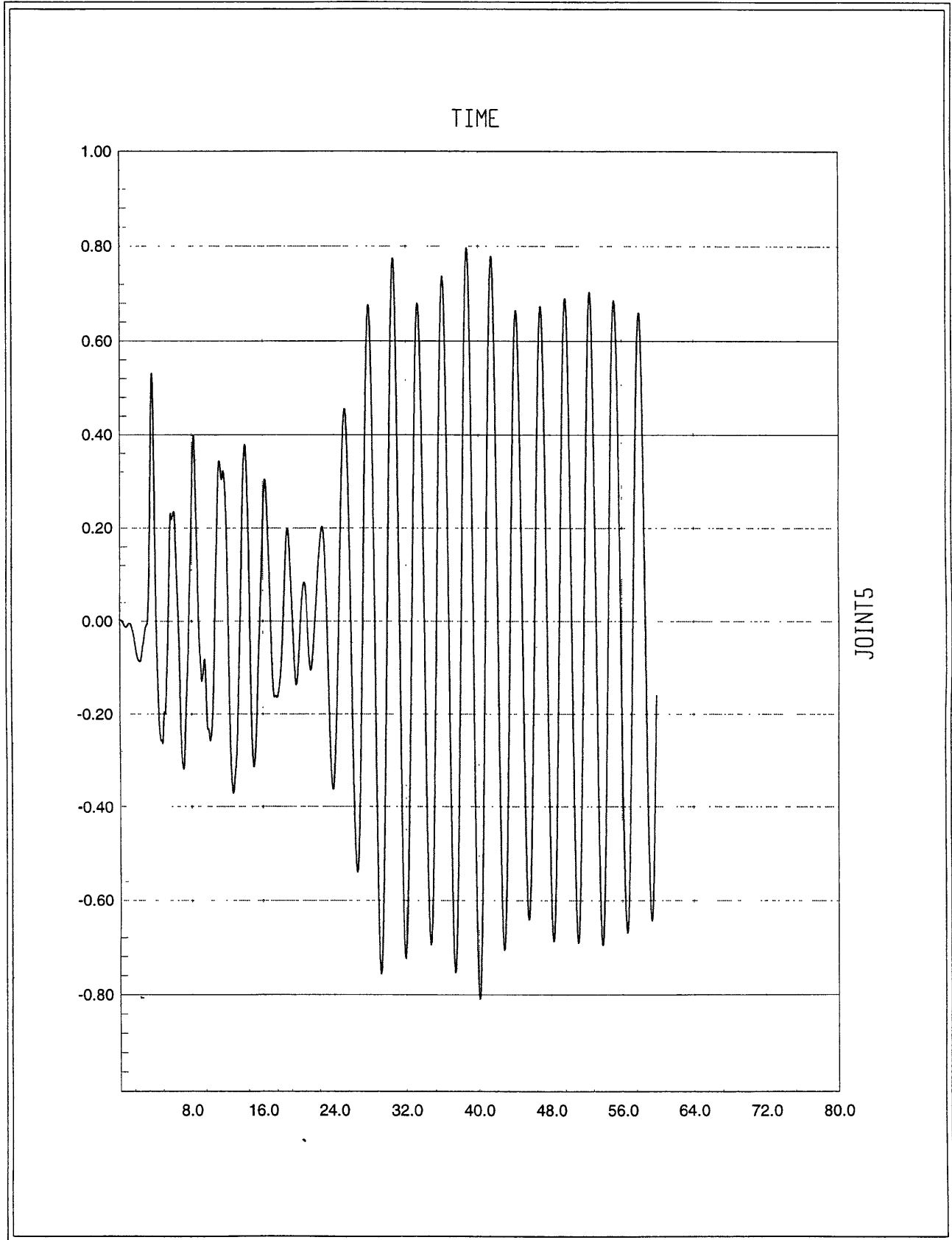




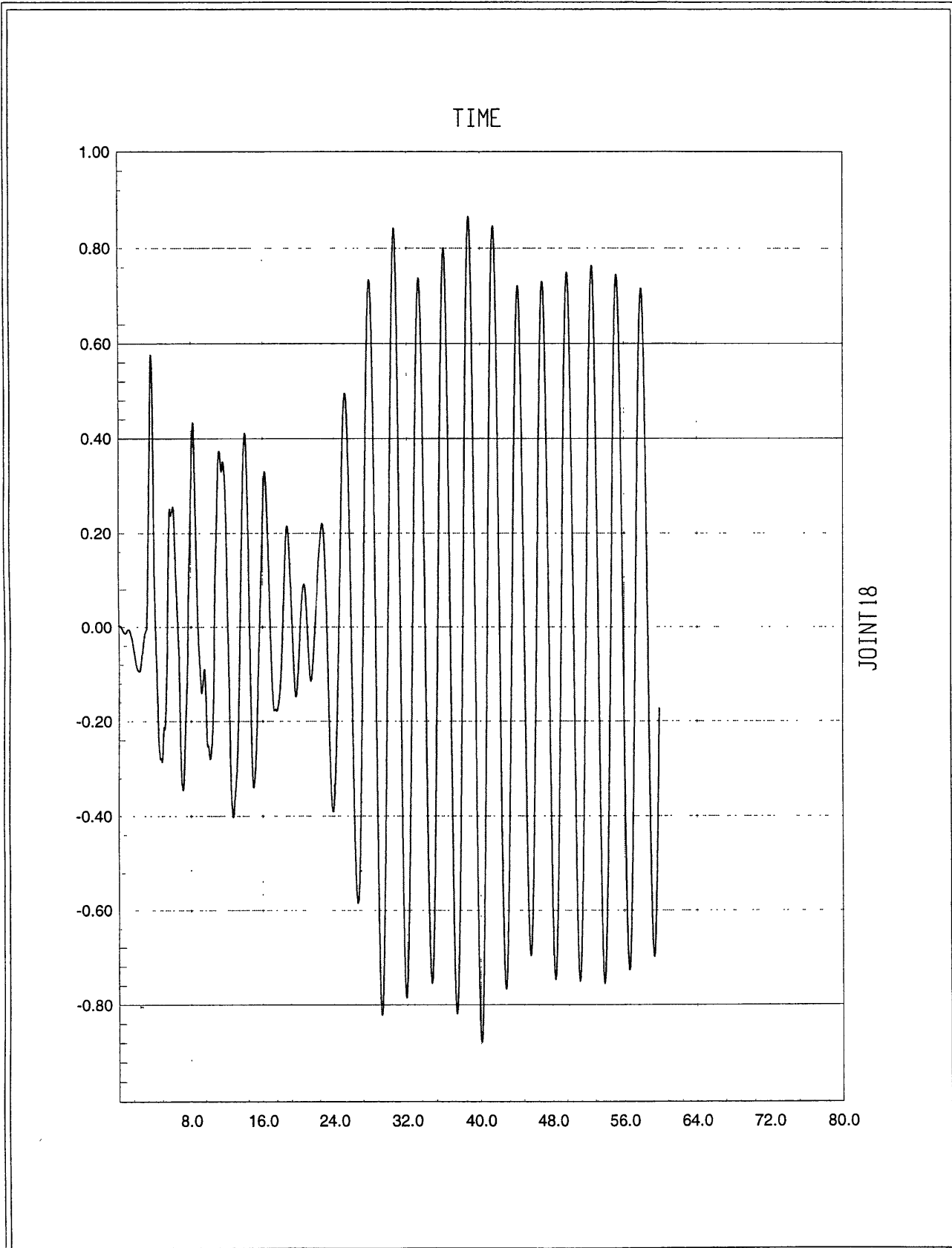
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Joint5: Joint 5 Displacement UX Vs Time
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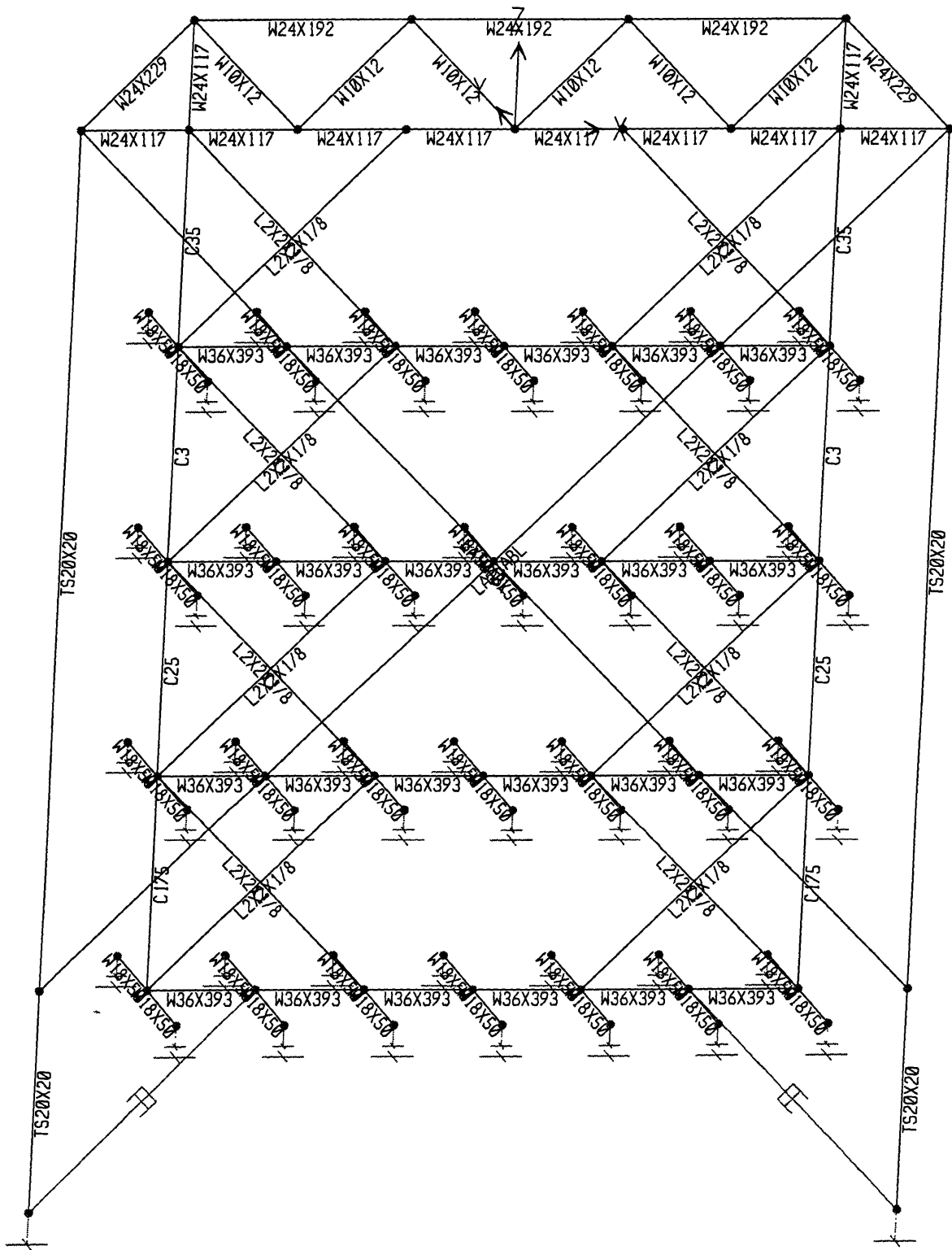
SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -2.036e+00 at 2.134e+01 Max is 2.032e+00 at 2.816e+01



SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -8.133e-01 at 4.0160e+01 Max is 7.954e-01 at 3.8820e+01



SAP2000 v6.11 - File:frame3 - Kip-ft Units
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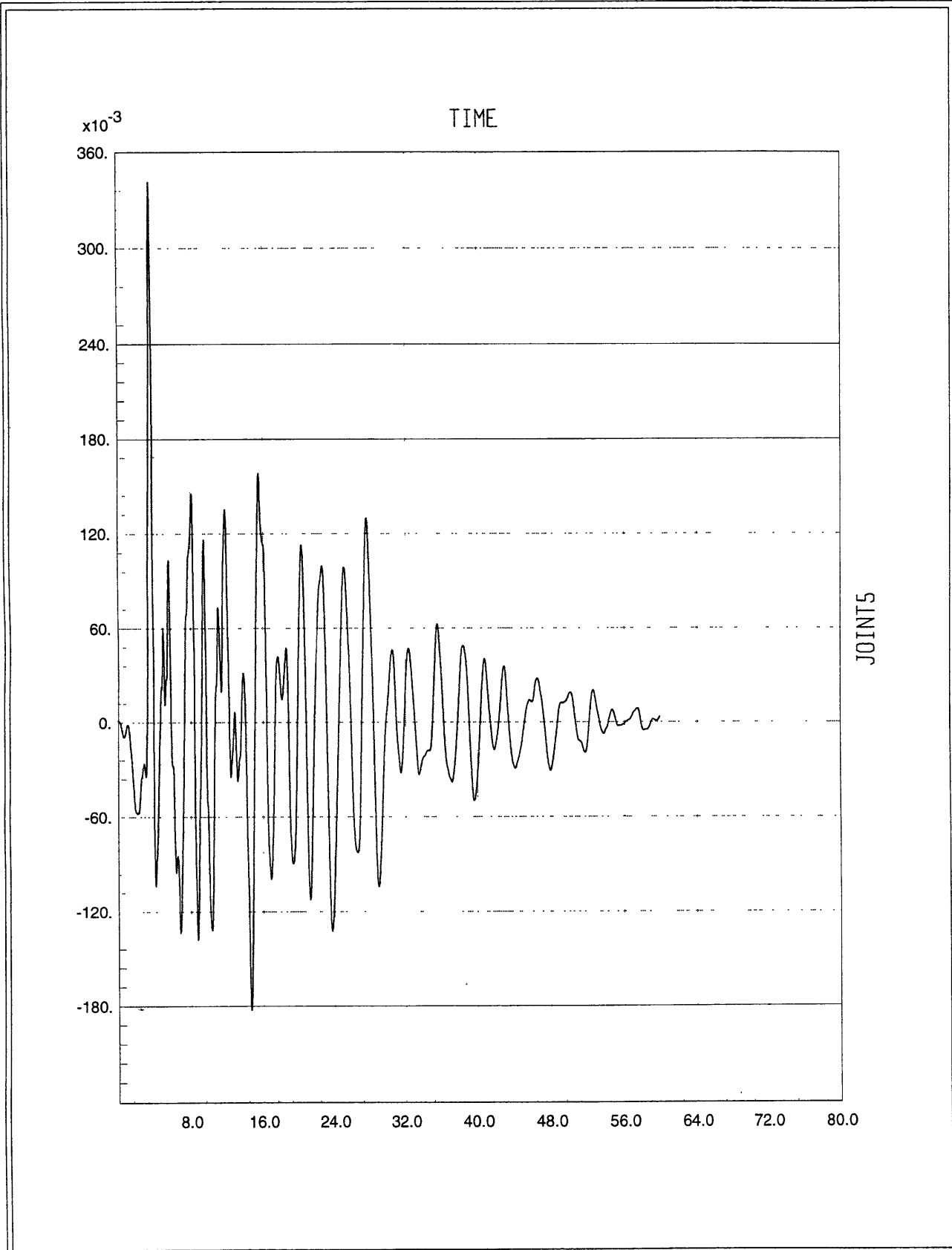
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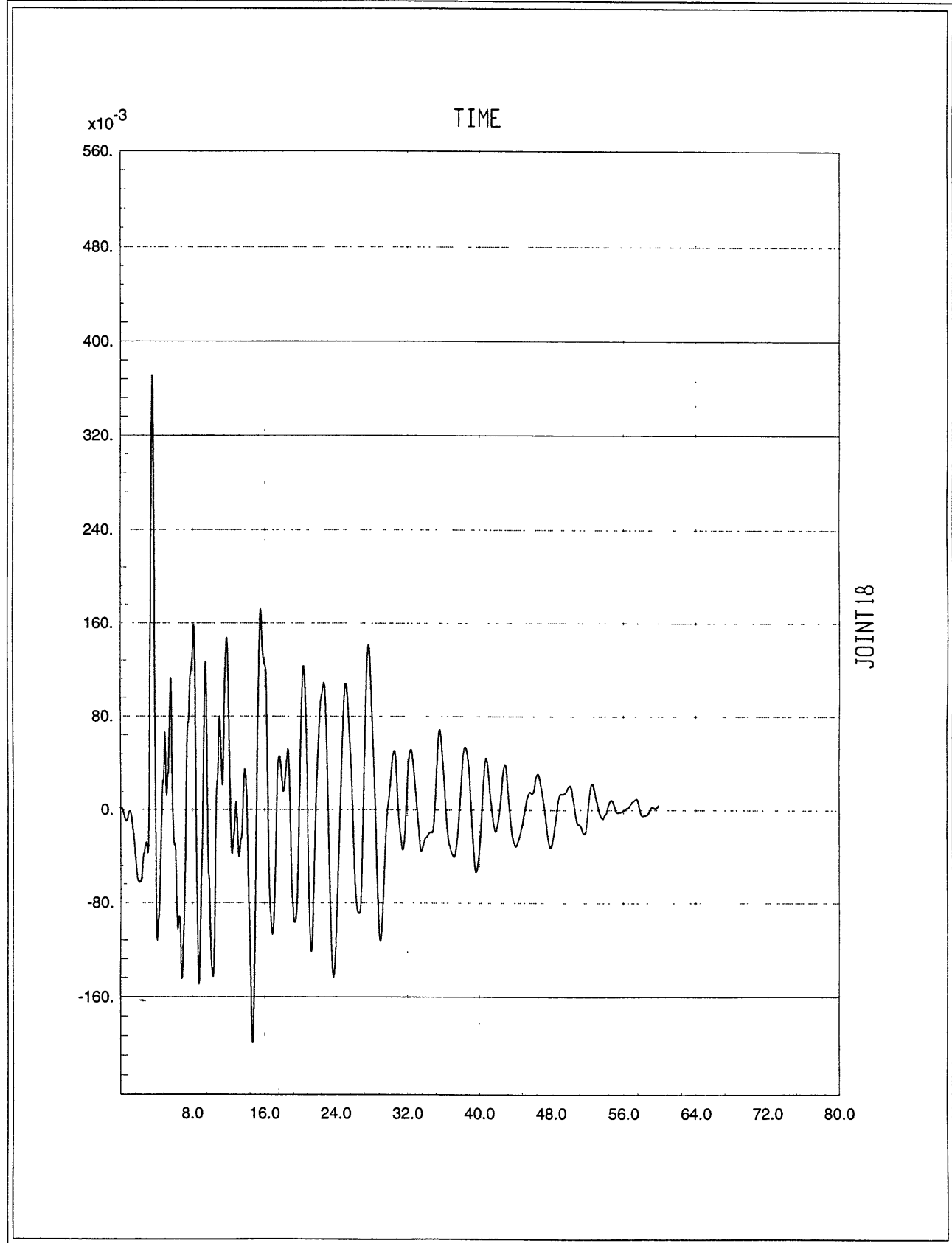
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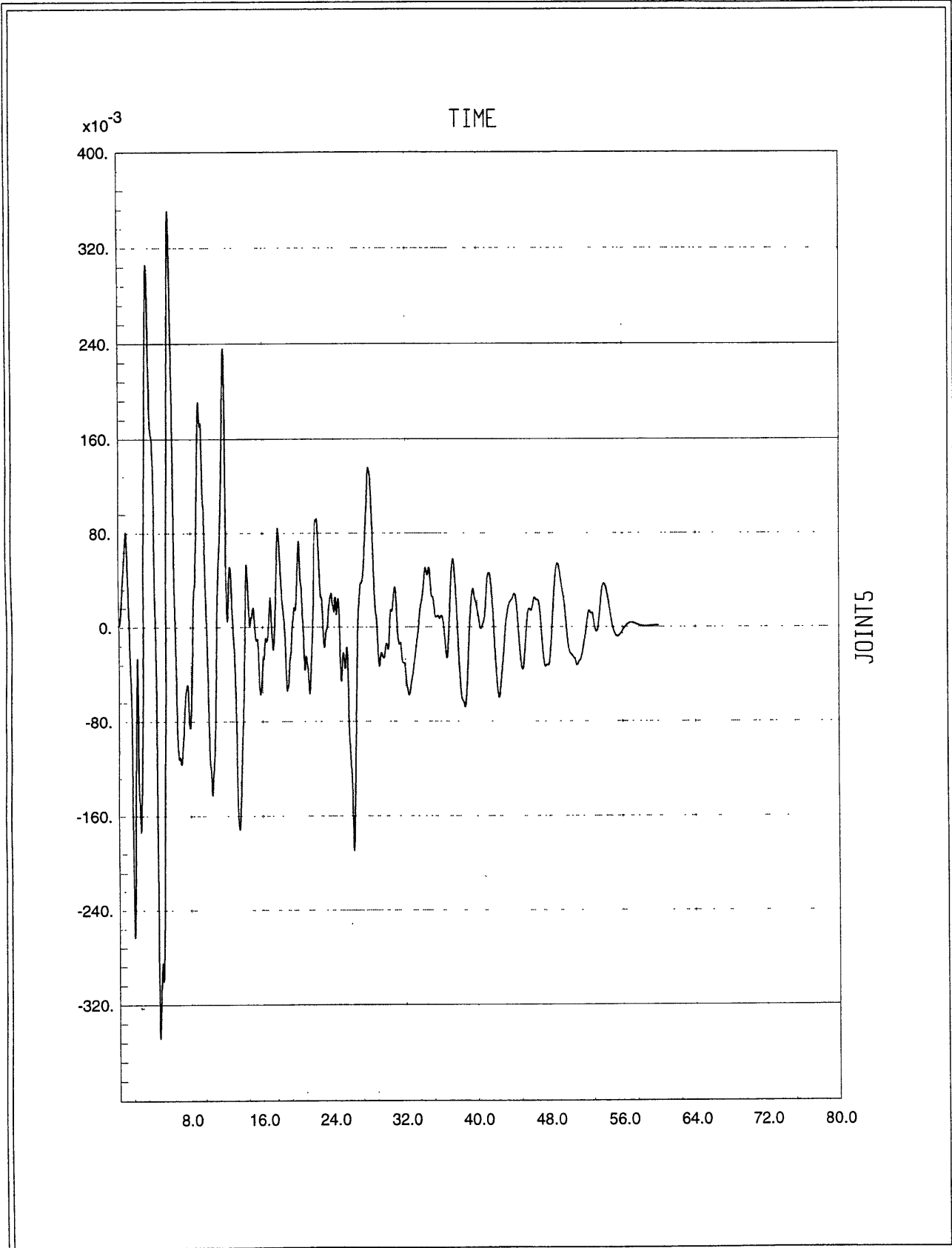
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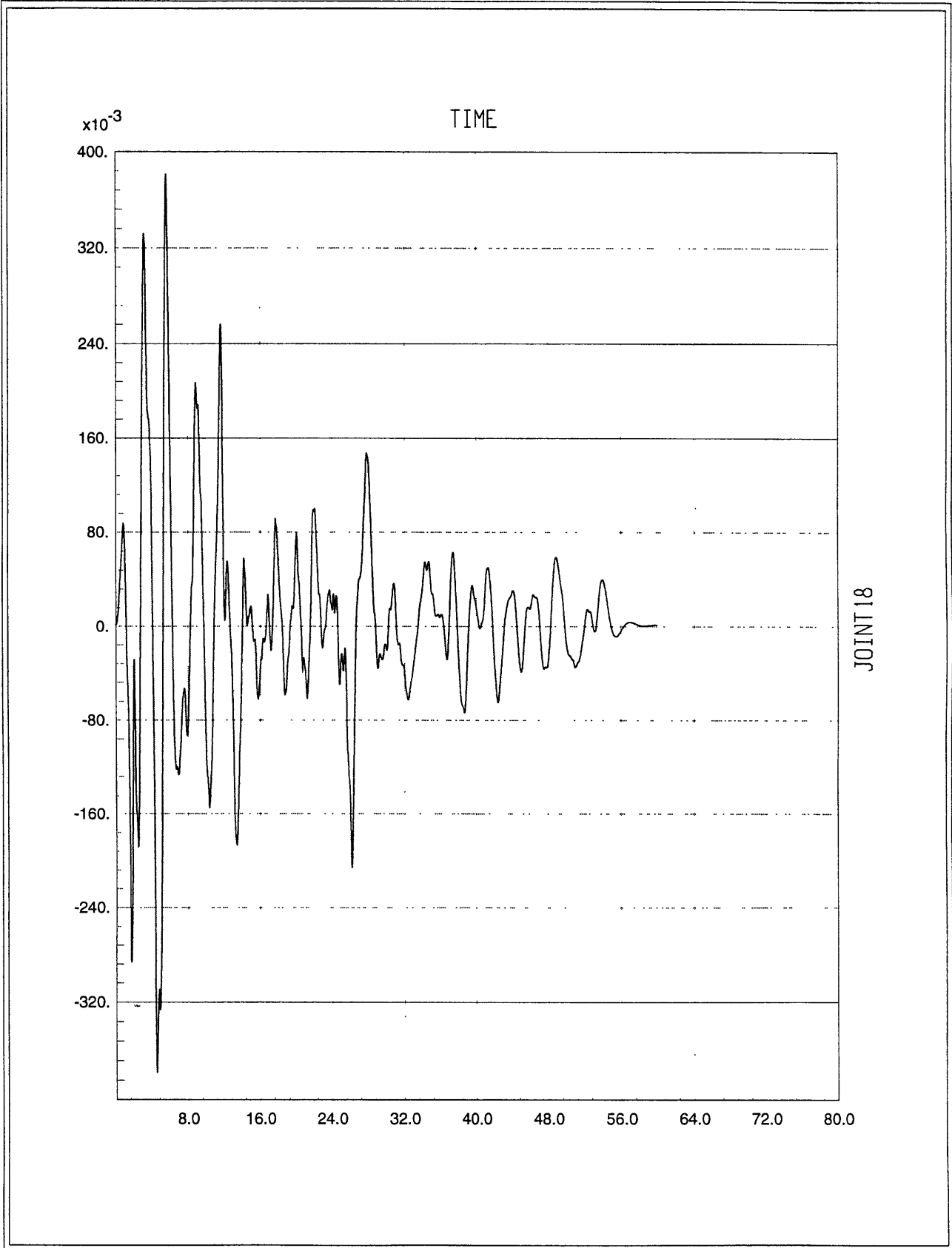
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.839e-01 at 1.4740e+01 Max is 3.412e-01 at 3.5800e+00



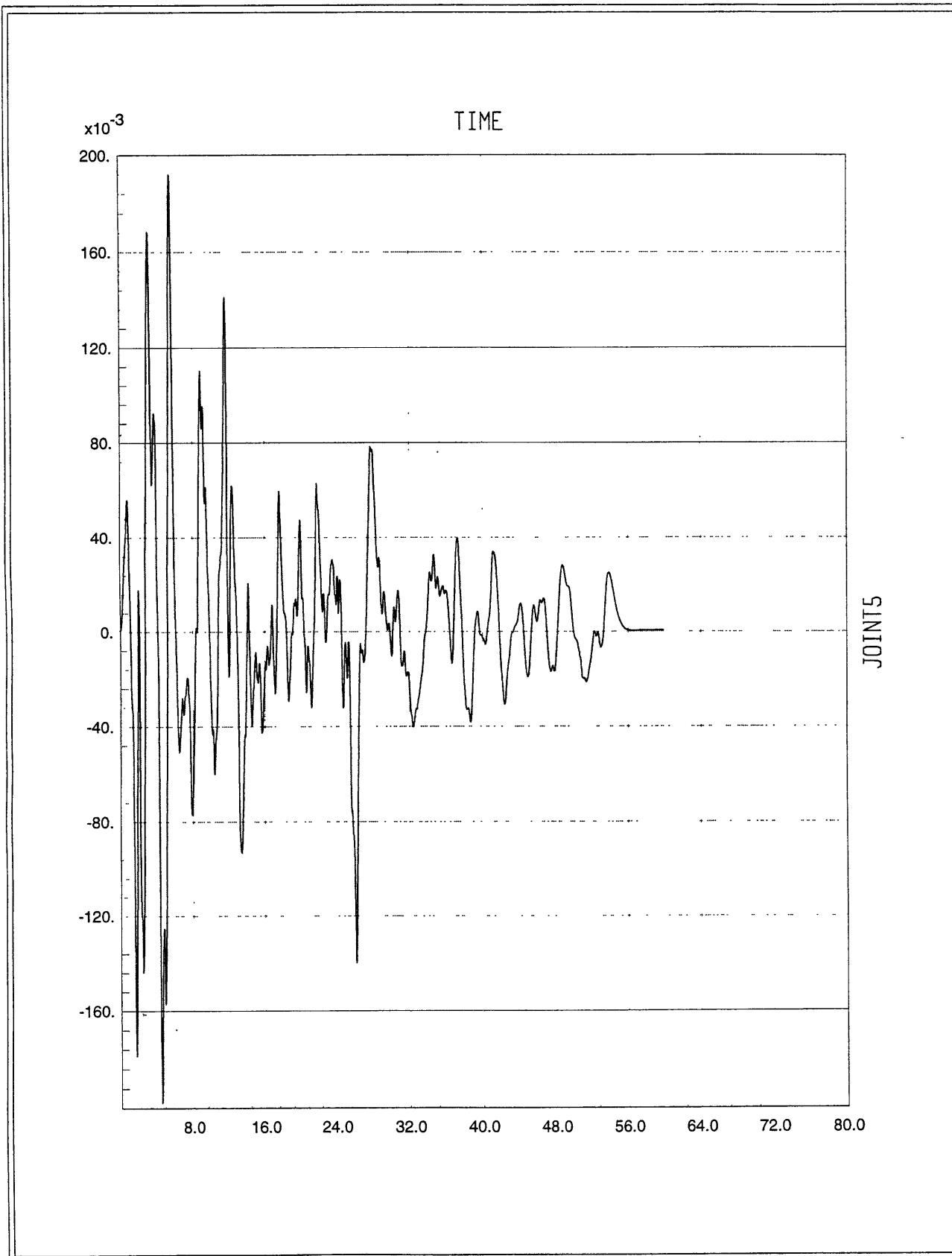
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -1.997e-01 at 1.4740e+01 Max is 3.705e-01 at 3.5800e+00



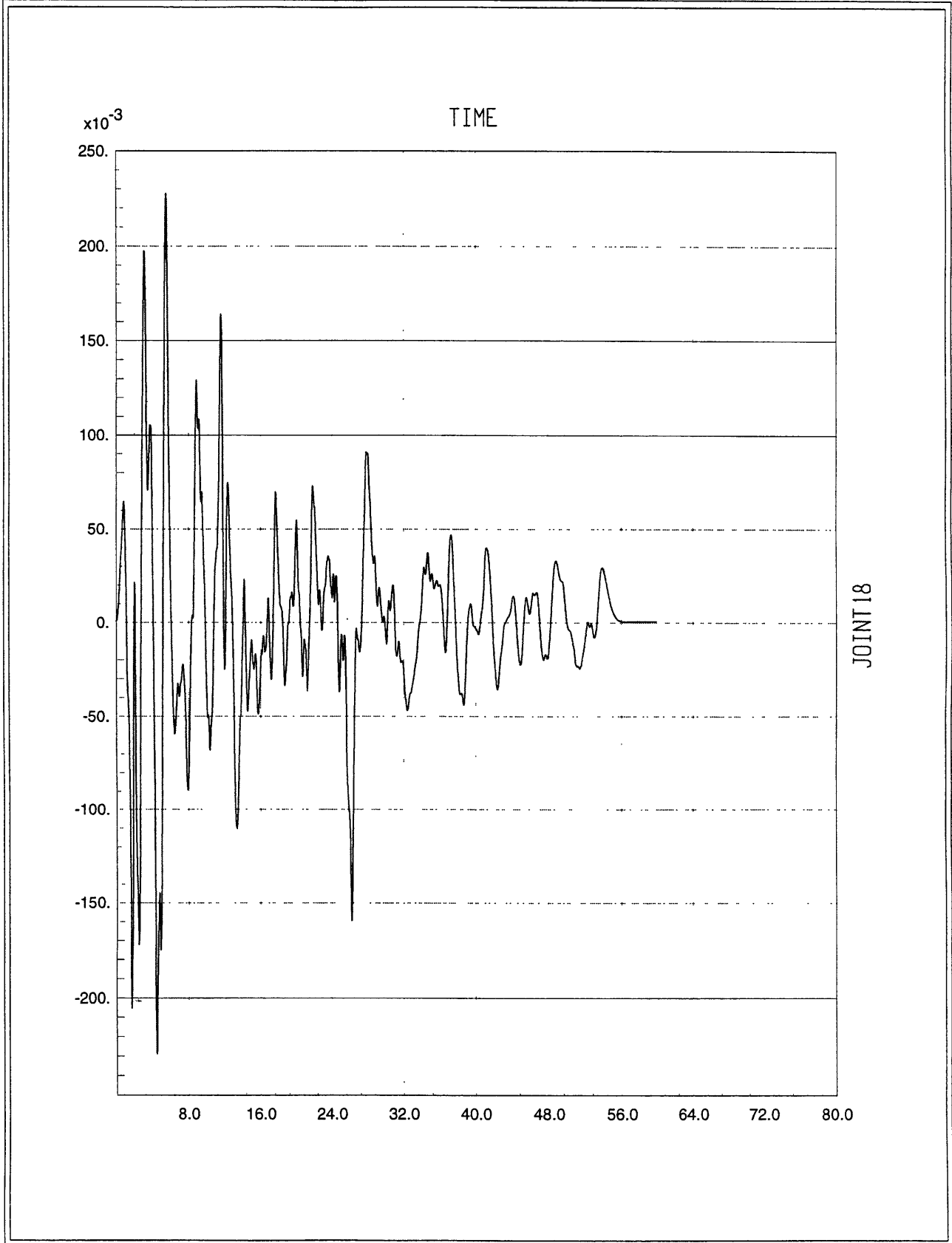
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -3.500e-01 at 4.5400e+00 Max is 3.506e-01 at 5.6400e+00



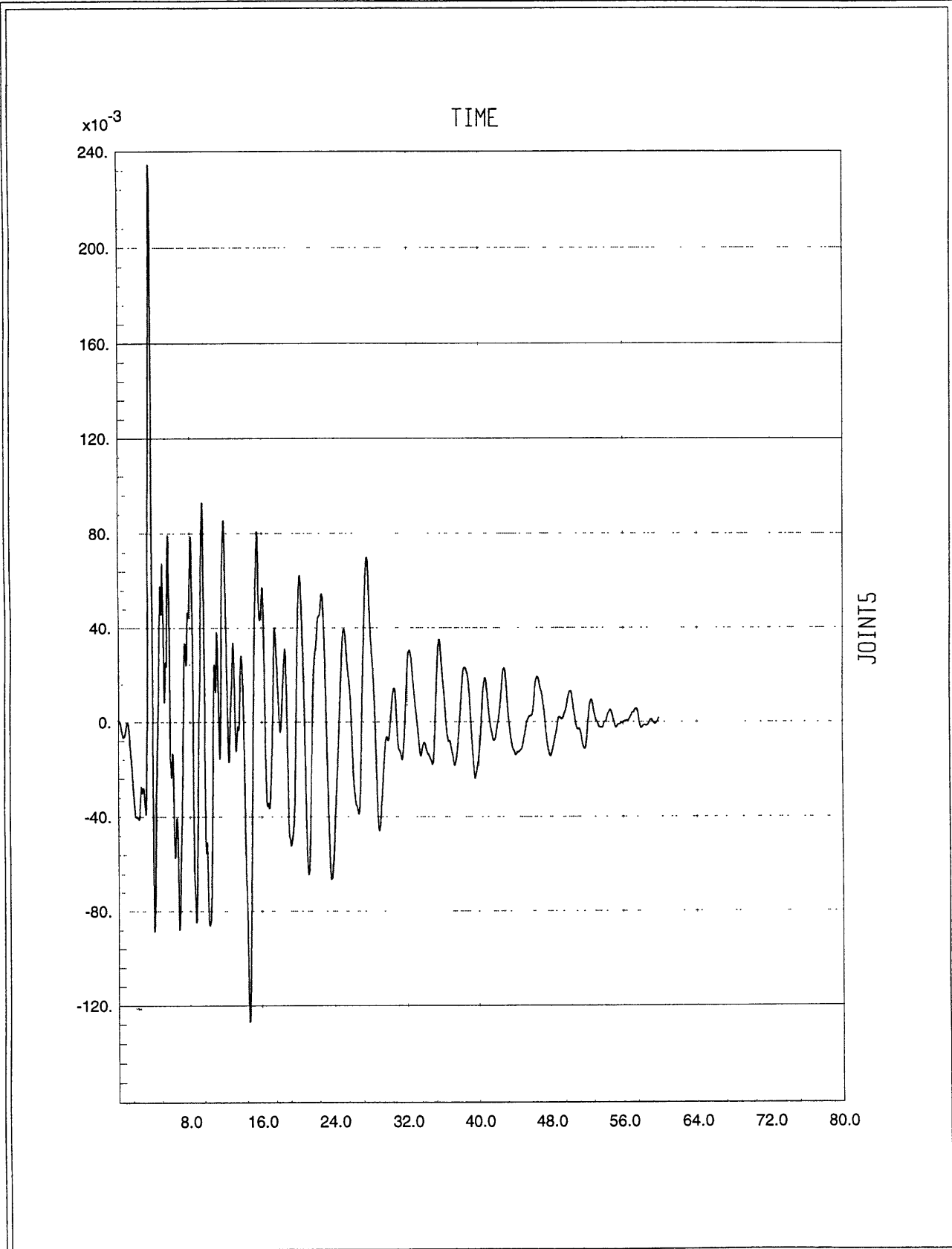
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
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Min is -3.801e-01 at 4.5400e+00 Max is 3.807e-01 at 5.6400e+00



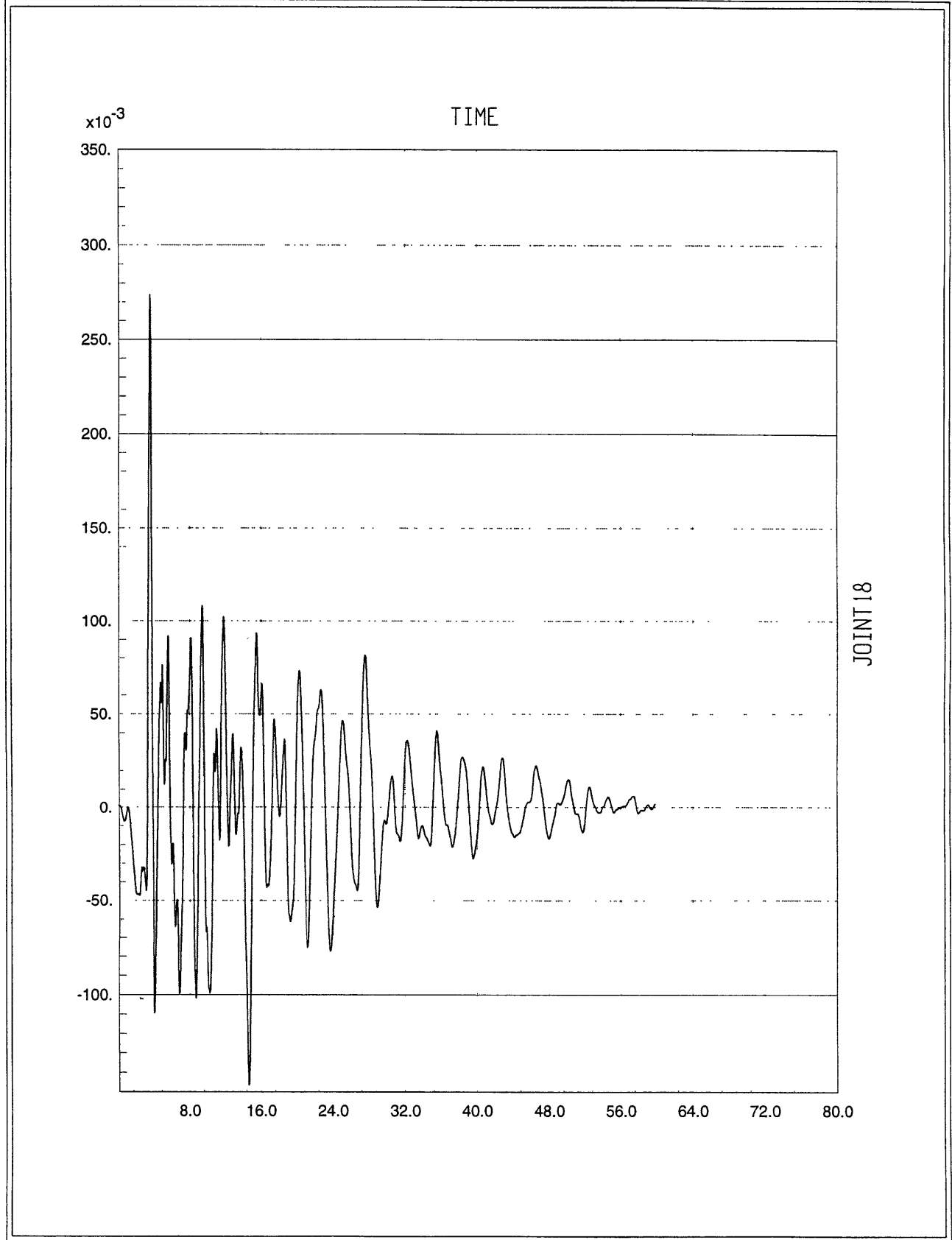
SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.988e-01 at 4.5000e+00 Max is 1.918e-01 at 5.6000e+00



SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -2.300e-01 at 4.5200e+00 Max is 2.272e-01 at 5.6000e+00



SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.279e-01 at 1.4700e+01 Max is 2.343e-01 at 3.5400e+00



SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -1.479e-01 at 1.4720e+01 Max is 2.735e-01 at 3.5600e+00

J. Cost Analysis Spreadsheet

CODE	ITEM	QUANTITY	UNIT	BARE COSTS \$				TOTAL INCL O&P
				MAT.	LABOR	EQUIP.	TOTAL	
051	STRUCTURAL METAL FRAMING							
051 200	Structural Steel							
051 250	Structural Steel Members, including bolted connections and erection							
051 250 0600	W 10x12	1273	L.F.	8,974.65	3,220.69	2,062.26	14,257.60	17,822.00
051 250 3700	W 18x50	10850	L.F.	303,800.00	26,257.00	13,345.50	343,402.50	429,253.13
051 250 5760	W 24x117	1500	L.F.	102,750.00	3,150.00	1,605.00	107,505.00	134,381.25
051 250 5788	W 24x192	900	L.F.	100,800.00	1,665.00	846.00	103,311.00	129,138.75
051 250 5789	W 24x229	424	L.F.	56,816.00	784.40	398.56	57,998.96	72,498.70
051 250 8109	W 36x393	3210	L.F.	735,090.00	6,837.30	3,466.80	745,394.10	931,742.63
051 220	Columns							
051 220 4658	20" x 8" 5/16" thick	720	LF	47,952.00	22,680.00	14,400.00	85,032.00	106,290.00
051 220 5659	20" x 20" 3/8" thick	2200	LF	187,000.00	69,300.00	44,000.00	300,300.00	375,375.00
051 500	Steel Wire Rope							
051 508	Wire Rope							
051 508 0059	3/4"	426	L.F.	660.30			660.30	825.38
051 508 0159	1-1/4"	1024	L.F.	4,392.96			4,392.96	5,491.20
051 508 0250	1-3/8"	219	L.F.	1,136.61			1,136.61	1,420.76
051 508 0251	1-1/2"	219	L.F.	1,355.61			1,355.61	1,694.51
051 508 0252	1-3/4"	600	L.F.	5,052.00			5,052.00	6,315.00
051 508 0253	2-1/2"	600	L.F.	10,320.00			10,320.00	12,900.00
051 508 0254	3"	440	L.F.	10,868.00			10,868.00	13,585.00
051 507 0255	3-1/2"	440	L.F.	14,828.00			14,828.00	18,535.00
053	METAL DECKING							
053 100	Steel Deck							
053 104	Metal Decking							
053 104 2100	galvanized, 1-1/2" deep, 22 gauge	28000	S.F.	22,680.00	5,600.00	560.00	28,840.00	36,120.00
053 104 0700	galvanized, 3" deep, 16 gauge	93000	S.F.	367,350.00	68,820.00	5,580.00	441,750.00	539,400.00
032	CONCRETE REINFORCEMENT							
032 200	Welded Wire Fabric							
032 207	Welded Wire Fabric							
032 207 0010	6x6-W1.4xW1.4	930	C.S.F.	6,789.00	11,811.00		18,600.00	1,162.50
033	CAST-IN-PLACE CONCRETE							
033 100	Structural Concrete							
033 126	Concrete, Ready Mix							
033 126 0300	3000 psi	232	C.Y.	12,412.00			12,760.00	14,036.00
033 126 0300	4000 psi	1004	C.Y.	55,220.00			55,220.00	60,742.00
033 126 1010	for structural lightweight, add		C.Y.	13,805.00			13,805.00	15,185.50
	Total =			2,070,052.13	220,125.39	86,264.12	2,376,789.64	2,923,914.30

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