EFFECTS OF GROUND MOTION SELECTION ON SEISMIC RESPONSE OF BUILDINGS

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ÖZGE KARAKÜTÜK

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submitted by **ÖZGE KARAKÜTÜK** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Gülbin Dural Ünver
Dean, Graduate School of **Natural and Applied Sciences** _____________________

Prof. Dr. Ahmet Cevdet Yalçıner
Head of Department, **Civil Engineering** _____________________

Prof. Dr. Ahmet Yakut
Supervisor, **Civil Engineering Department, METU** _____________________

Examining Committee Members:

Prof. Dr. Murat Altuğ Erberik
Civil Eng. Dept., METU _____________________

Prof. Dr. Ahmet Yakut
Civil Eng. Dept., METU _____________________

Assoc. Prof. Dr. Ayşegül Askan Gündoğan
Civil Eng. Dept., METU _____________________

Assoc. Prof. Dr. Eray Baran
Civil Eng. Dept., METU _____________________

Dr. Nazan Kılıç
AFAD Afet ve Acil Durum Yönetimi Başkanlığı _____________________

Date: 06.02.2015

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name: Özge Karakütük

Signature :

ABSTRACT

EFFECT OF GROUND MOTION SELECTION ON SEISMIC RESPONSE OF BUILDINGS

Karakütük, Özge

M.Sc., Department of Civil Engineering
Supervisor: Prof. Dr. Ahmet Yakut

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In this study, effects of ground motion selection and scaling procedure given in different earthquake design codes were investigated. To observe differences between the scaling procedures defined in various seismic design codes, these procedures are applied employing the response spectra defined in these specifications.

Four reinforced concrete moment resisting frame buildings were used in the analyses. These buildings have 3, 4, 6 and 8 stories. This way, effects of scaling on low-rise and mid-rise buildings were studied.

Material properties, structural member dimensions and span lengths of buildings were selected by considering the properties of the constructed reinforced concrete buildings in Turkey. Reinforcement of these buildings were determined with respect to TS 500. Two dimensional finite element models of these buildings were developed and necessary information for scaling such as fundamental period was obtained. Then, ground motion sets were selected and scaled for each building and each spectrum according to each procedure employed.

Scaling procedures used and compared in this study are based on TEC-07, ASCE/SEI 7-10 and Eurocode-8. According to these specifications both response spectrum and time history analyses of the structures were performed. The results of these analyses were used to evaluate the effect of ground motion scaling on seismic response of low- and mid-rise reinforced concrete moment resisting frame buildings.

Due to the criteria on scaling of ground motion recordings in all earthquake design codes used in this research, time-history analyses yielded more conservative results than response spectrum analyses. Scaling according to TEC-07 or Eurocode-8 procedures gave closer results to response spectrum analyses as compared to scaling with respect to ASCE/SEI 7-10. The difference between time history analyses and response spectrum analyses diminishes as the number of story increases. In other words, as the fundamental period of the structure increases the results of response spectrum and time-history design procedures converge.

Keywords: RC moment resisting frame structures, ground motion scaling, response spectrum, code-based scaling, time history analyses.

ÖZ

DEPREM KAYIT SEÇİMLERİNİN BİNALARIN SİSMİK DAVRANIŞLARI ÜZERİNDEKİ ETKİSİ

Karakütük, Özge

Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Bu çalışmada farklı deprem şartnamelerine göre yapılan yer hareketi seçimi ve ölçeklendirmesinin bina tasarımına etkileri incelenmiştir. Ayrıca, deprem kodlarında belirlenmiş olan ölçeklendirme yöntemlerinin birbiri arasındaki farklılıkların daha iyi anlaşılabilmesi adına aynı ölçeklendirme yöntemi, her şartname için ayrı ayrı uygulanmıştır.

Çerçeve sistemine sahip, sırasıyla 3, 4, 6 ve 8 katlı 4 betonarme bina modellenmiştir. Binaların kat adetlerinden de anlaşılabileceği gibi yer hareketi ölçeklendirmesinin az katlı ve orta katlı yapılar üzerindeki etkileri araştırılmıştır.

Binalar modellenirken kullanılan malzeme sınıfları, yapısal eleman boyutlandırmaları ve aks açıklıkları Türkiye'de alışılagelmiş betonarme bina tasarımları göz önünde bulundurularak gerçekleştirilmiştir. Yapısal elemanların donatıları TS-500'e göre hesaplanmıştır. Bu yapıların sonlu eleman analizlerinin yapılabilmesi adına boyutlu modelleri oluşturulmuş ve yer hareketi ölçeklendirmesi için gerekli olan, bina ana periyodu, hedef spektral değerler gibi bilgiler elde edilmiştir. Elde edilen bu sonuçlarla birlikte her prosedüre göre, her bina ve her spektrum için yer hareketi kayıtları seçilmiş ve ölçeklendirilmiştir.

TEC-07, ASCE/SEI 7-10 ve Eurocode-8 bu çalışmada kullanılan deprem şartnameleridir. Davranış spektrumu yöntemiyle ve zaman tanım alanında yapılan analizler bu standartlara göre uygulanmıştır. Yapılan bina analizlerinin sonuçları, az ve orta katlı betonarme çerçeve sistemli binalar için yer hareketi ölçeklendirmesinin etkileriyle ilgili bilgileri vermiştir.

Bu çalışmada kullanılan tüm deprem şartnamelerinde yer hareketi ölçeklendirmesi için ciddi sınırlandırmalar bulunduğundan dolayı, zaman tanım alanında yapılan analizler, davranış spektrumu analizlerinden daha konservatif sonuçlar vermiştir. TEC-07 veya Eurocode-8 göre yapılan yer hareketi ölçeklendirmeleri, davranış spektrumu analizleri sonuçlarına ASCE/SEI 7-10'a göre yapılan ölçeklendirmelerden daha yakın analiz sonuçları vermiştir. Bina kat sayısı arttıkça zaman tanım alanında yapılan analizler ve davranış spektrumu analizleri birbirine daha yakın sonuçlar vermektedir. Bir başka deyişle, yapıların temel periyotları büyüdükçe, deprem analiz sonuçları birbirine benzemektedir.

Anahtar Kelimeler: Betonarme çerçeveli yapılar, yer hareketi ölçeklendirmesi, ivme spektrumu, koda bağlı ölçeklendirme, zaman alanında tanımlı deprem analizi, sismik talep.

To My Family…

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Last but not the least I would like to thank my family. They are the meaning of my life. Words cannot express how grateful I am to them for all of the sacrifices that they have made on my behalf. They are the only reason what gets me thus far. Without their help and support, this thesis would neither start nor come to an end. At the end, I would like to express my sincere thanks also for my one and lovely little sister Bilge, without her help and support this thesis would not end. Thanks for all of the contributions made to me.

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$A_{0,g}$:	effective ground motion acceleration (g)
ASCE:	response spectrum defined in Minimum Design Loads for Buildings and Other Structures of American Society of Civil Engineers Standards
EC:	type 1 response spectrum defined in Eurocode 8 : Design of Structures for Earthquake Resistance
g:	acceleration of gravity (m/s^2)
i:	number of the story
MoF:	mechanism of fault
M_w:	moment magnitude
PGA:	peak ground acceleration
PGD:	peak ground displacement
PGV:	peak ground velocity
R_{jb}:	Joyner-Boore distance, horizontal distance to the surface projection of the rupture plan (km)
R_{rup}:	rupture distance, closest distance to the rupture plane (km)
S_a:	spectral acceleration
$S_{a,yield}$:	spectral acceleration at yielding

S_d: spectral displacement

$S_{d,target}$: target spectral displacement

T: period (s)

T_1: fundamental period of the structure (s)

TEC: response spectrum defined in Turkish Earthquake Code 2007.

Tr: return period of earthquake

$V_{s,30}$: average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less

w_i: weight of i^{th} floor (t)

α: ratio of the design ground acceleration to the acceleration of gravity

ε: dispersion of spectral acceleration

ϕ_i: mode shape at i^{th} floor

CHAPTER 1

INTRODUCTION

1.1 General

In many countries and regions of the world, earthquakes are one of the most common natural disasters, which affect both human life and property. To avoid negative effects of earthquakes, structures should be designed to resist earthquakes.

There are two main types of earthquake resistance design method in most of the codes, which are linear and non-linear procedures. Linear methods have been preferred, since they are easy to apply to structures and also non-linear methods require a significant amount of acknowledgement, time and effort.

It is obvious and clear that the earthquake design of any structure must satisfy requirements of the relevant earthquake codes. Whether applying linear or non-linear earthquake analyses, code-defined spectra, user-defined spectra or ground motion records can be used. If ground motion records are going to be preferred, they are required to be scaled according to the earthquake codes.

Although there are different opinions about scaling procedures, it is intended in this study to investigate code based scaling procedures and their effects on low and mid-rise buildings.

1.2 Study Cases

In this study, scaling procedures in Turkish Earthquake Code 2007, Eurocode-8 and ASCE7-10 were compared. Four different reinforced concrete buildings are used to evaluate effects of ground motion scaling procedures in different period ranges. The buildings are modeled considering the common properties of buildings used for residential occupancy in Turkey. These buildings have three, four, six and eight floors respectively. All of them were first modeled in Probina. It was assumed that the buildings have rigid diaphragm floors; concrete class is taken as C20 or C25 and reinforcement steel class was S420. The reinforcement details of buildings were designed according to Turkish standards. The soil type was taken as Z3 in TEC2007, C in Eurocode-8 and ASCE7-10. By limiting the properties of earthquake records such as magnitude, distance and soil type, 51 ground motion recordings were selected as candidate. Then, according to their faulting mechanisms and their response spectra, five ground motion sets were obtained. Among these, the most suitable ground motion set was selected and three subsets were generated for frame analyses. Linear time history analyses of the frames were conducted by utilizing these three ground motion record sets. Response spectrum analyses of the frames were executed by using response spectra in TEC, ASCE and EC8, respectively.

1.3 Data and Software Used in the Study

The building information data was obtained from the study entitled “A Statistical Study on Geometrical Properties of Turkish Reinforced Concrete Building Stock” (Azak et.al., 2014).

The ground motion records were selected from Pacific Earthquake Engineering Research Center database (http://peer.berkeley.edu/smcat/), with respect to the earthquake characteristics intendedsuch as moment magnitude, Rjb, and soil conditions.

The reinforcement details of buildings were calculated according to Turkish Standards including TS500, TS708, and TS498.

The building models were prepared in Probina and Seismostruct softwares. Probina is a package software developed by Prota, which makes three dimensional model and design reinforced concrete buildings with respect to Turkish Standards. Seismostruct is another software produced by Seismosoft, in which both linear and non-linear analyses of structures can be conducted both in two and three dimensions. Scaling of ground motion records is done in Seismosignal software which is also developed by Seismosoft.

1.4 Objectives and Scope of the Study

The objectives of the presented study is to investigate the effects of selection of ground motions and also changes in scaling procedures with respect to different earthquake codes on the design of low and mid rise buildings.

The following steps were applied to achieve the objective of the study:

1. Based on modeling and design of four different reinforced concrete buildings in Probina, case study buildings are obtained.
2. According to the data taken from Probina, Seismostruct frame models were created.
3. To obtain period information Eigenvalue analyses were performed in Seismostruct.
4. Ground motion records, which were used in time history analyses, were selected according to the criteria on the magnitude, distance and soil type properties and the restriction of the codes.
5. Five different ground motion sets were formed from the candidate ground motion records.
6. The most suitable ground motion set was selected and three subsets were formed for scaling.

7. Two scaling methodologies were applied to see the effects of different types of scaling.
8. The selected ground motions were scaled with respect to each procedure defined in earthquake codes by using Seismosignal software for each building. To see the effects more clearly, each scaling procedure was applied to all spectra in the codes.
9. By applying scaled ground motion accelerations to the frame models in Seismostruct, time history analyses were performed.
10. Response spectrum analyses of each frame were executed using TEC, ASCE and EC8 spectra, separately.
11. According to the results of time history analyses, the effects of the selection of ground motion records and the scaling procedures on earthquake response of low-rise and mid-rise buildings were examined.

The steps of the study are also shown as a flowchart in Figure 1.1.

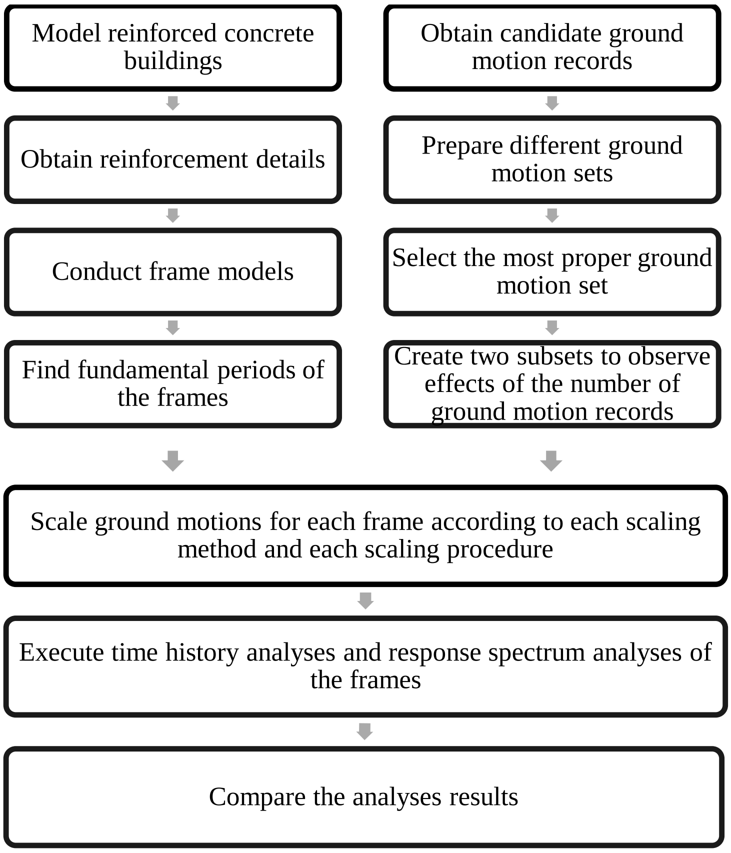

Figure 1.1. Flowchart of the Study

1.5 Organization of Thesis

This thesis is composed of five chapters. The first chapter gives a general overview of the study.

Chapter 2 includes a literature survey on scaling procedures in three different earthquake codes for use in design, and other scaling options in literature.

Information about reinforced concrete building/frame models and their design and analysis results needed for scaling are given in Chapter 3.

Chapter 4 presents selection and scaling of ground motion records with respect to criteria defined in the earthquake codes. Additionally, response spectrum and time history analysis results are examined in this chapter.

Chapter 5 presents summary and conclusion regarding the effects of scaling procedures on response of low rise and mid-rise reinforced concrete buildings.

CHAPTER 2

LITERATURE SURVEY AND SCALING IN CODES

2.1. Introduction

Instead of using traditional seismic analysis methods, more advanced methods like response spectrum and time history analysis are more commonly preferred. This trend is mainly due to the developed analysis softwares and high technology, which allow performing complex calculations and iterations easily.

Haselton et. al. (2012) defines the reasons of choosing response history analysis instead of other analysis types in three main topics:

- More precise calculation methods such as finite element should be utilized to make the analyses.
- Material properties of structural elements are defined in more detail in design
- Due to previous statements, this type of seismic analysis gives more realistic and precise results.

When seismic design and analysis of a structure is conducted, code-defined spectra, user defined spectra or ground motion records can be used. To be able to apply response spectra compatible ground motion recording on the structure, some scaling procedures should be followed.

Several researchers in different studies (Carballo and Cornell 2000; Silva and Lee 1987; Bolt and Gregor 1993) recommend using frequency domain methods to achieve necessary ground motion set. On the other hand, some other researchers like Naeim (1999) proposed to utilize time domain methods and change only the amplitude of recorded earthquake ground motions. In addition to this Baker (2007) states that it is preferred to select ground motion recordings and scale them to match necessary intensity level by multiplying their amplitude. Because of the lack of ground motion record libraries, scaling seems to be used in common practice.

O'Donnell et. al. (2013) express that selection of ground motions for scaling is mainly based on rupture distance, site conditions and magnitude of the expected event, which is used in seismic design of the structure.

In this study, three code required procedures for scaling of response spectrum compatible ground motions were employed. The codes employed are Turkish Earthquake Code (2007), Eurocode 8 (1998) and ASCE 7-10 (2010).

2.2 Selection and Scaling in Turkish Earthquake Code

According to Turkish Earthquake Code 2007, three methods of seismic analysis can be used. These methods are equivalent lateral force method, response spectrum analysis and time history analysis.

When response spectrum analysis is performed, the code-defined spectrum or user-defined spectrum can be used. For time history analysis, proper earthquake acceleration data that is compatible with the spectrum must be selected and scaled. For the ground motion selection and scaling, the following limitations should be considered:

- At least three earthquake ground motions, whether real or artificial, must be used.
- The record duration of strong motion must be five times greater than the fundamental period of the structure and also more than 15 seconds.

- The mean of acceleration values of selected ground motions for zero period shall be larger than the effective ground motion acceleration coefficient times gravity acceleration (A_0.g).
- The mean spectral accelerations of selected ground motions for 5% damping ratio must be more than 90 percent of code-defined response spectrum between $0.2T_1$ and $2.0T_1$ where T_1 is the fundamental period of the building,.
- If linear time history analysis is to be performed, then ground motion spectral accelerations shall be reduced as defined in Eq 2.1.

$$S_{aR}(T_n) = \frac{S_{ae}(T_n)}{R_a(T_n)} \qquad \text{Eq.2.1}$$

- Seismic analysis shall be performed in the time domain.
- If three ground motions are used, the maximum of the analysis result is considered. Whereas, if at least seven records are used, the mean of the results should be considered for the design.

Figure 2.1 shows zero period acceleration (A_0.g) and the range of period required for scaling according to TEC2007. The shaded area in the Figure 2.2 shows the range where spectral accelerations of scaled ground motions shall be higher than 90% of code-defined elastic spectrum.

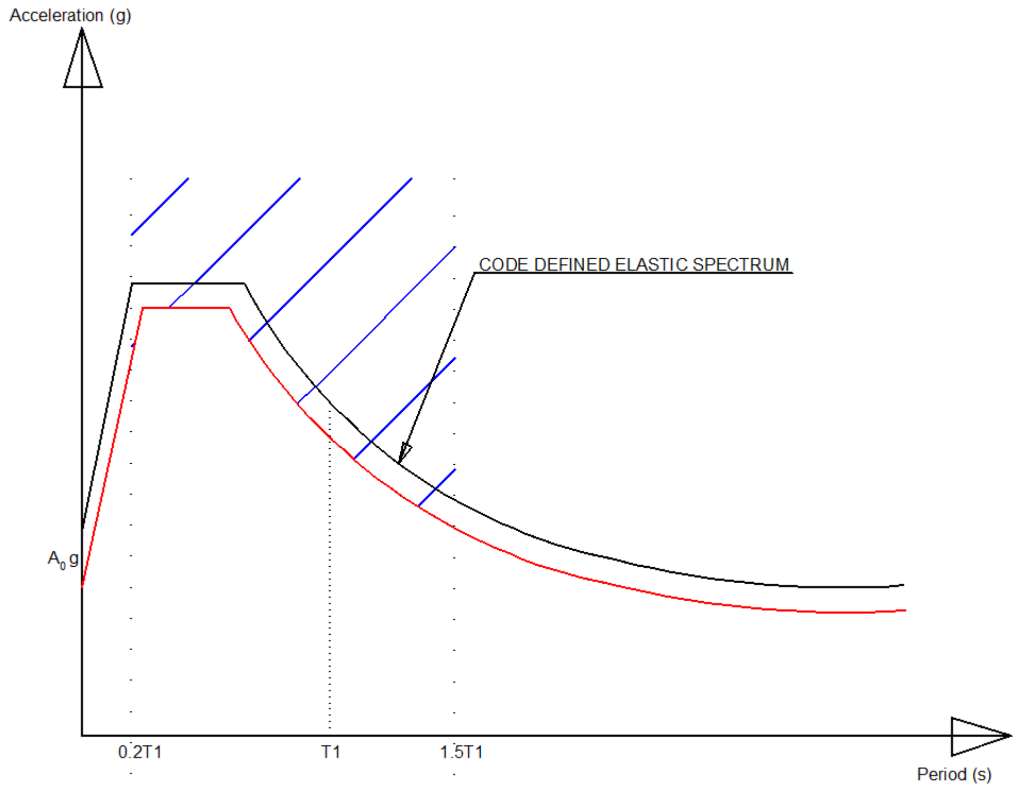

Figure 2.1. Response Spectrum Properties Used in Scaling According to TEC2007

2.3 Selection and Scaling in Eurocode 8

Similar to Turkish Earthquake Code, Eurocode 8 uses three analysis procedures; force method, response spectrum analysis and time history analysis. There are two types of response spectra defined in Eurocode 8, Type 1 and Type 2 where Type 2 is recommended to be used if the surface-wave magnitude of the earthquakes that contribute most to the seismic hazard defined for the site is not greater than 5.5. Because earthquakes which have magnitude greater than 5.5 are used here, Type 1 spectrum is utilized in this study.

Recommendations for time history representation of the seismic action in Eurocode 8 can be listed as follows:

- At least three simultaneous ground motion records must be selected, if user-defined response spectrum analyses or time history analyses are desired, similar to Turkish Earthquake Code.
- Average of zero period acceleration of records must be greater than design ground acceleration on type A soil times soil factor (a_g.S) defined by the code.
- Unlike TEC2007, minimum duration of the strong part of the accelerograms does not depend on the fundamental period of the structure and must be equal or greater than 10seconds.
- No value of 5% damped mean elastic spectrum of scaled ground motion records must be lower than %90 of code-based 5% damped response spectrum, in the range of $0.2T_1$ to $2.0T_1$.
- Same as in Turkish Earthquake Code, if three ground motions are used, the most unfavorable value of the analysis result is considered. On the other hand, if at least seven records are used, the average of the results should be used for the design.

2.4 Selection and Scaling in ASCE 7-10

Selection and scaling procedure in ASCE7-10 has some similarities but also some differences with Turkish Earthquake Code and Eurocode 8. The provisions in ASCE7-10 are as follows:

- Minimum of three appropriate earthquake ground motions must be used.
- If two dimensional analyses are implemented, each ground motion shall be selected from actual recorded event and consist of a horizontal acceleration history.
- Unless there is not any suitable recorded ground motion available, then appropriate simulated ground motions can be used.
- The ground motions shall be scaled such that the average value of 5% damped response spectra of the selected records should be greater than the code-based

response spectrum between 0.2T and 1.5T where T is the fundamental period of the structure.

- There is no obligation about acceleration of ground motion records at zero period.
- If there is less than seven records selected, maximum of them is used, and if at least seven records are used, the mean of the results can be considered for the design like TEC and EC.

Scaling requirements of selected ground motions according to ASCE 7-10 are shown in Figure 2.2. Spectral accelerations of scaled ground motions shall be higher than code-defined elastic spectrum in the given period range, which is also shown as shaded area in Figure 2.2.

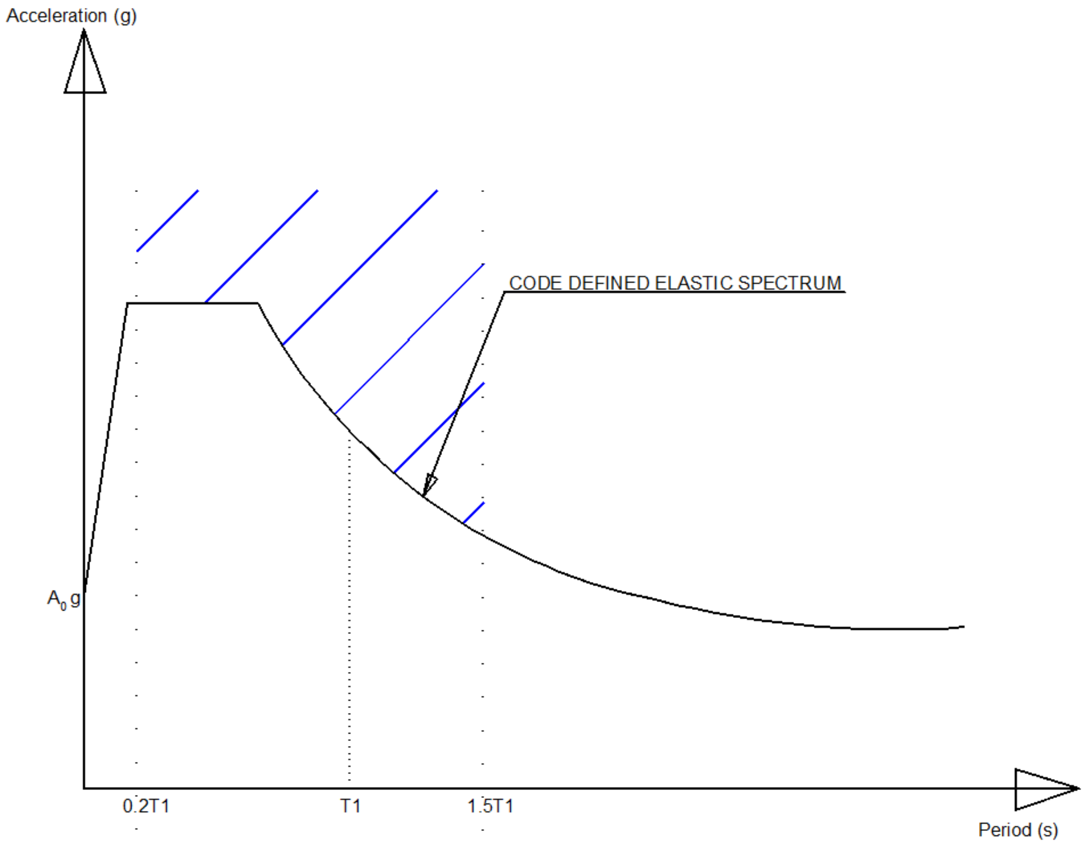

Figure 2.2. Response Spectrum Properties Used in Scaling According to ASCE

2.5 Discussion on Code Procedures

The differences in selection and scaling procedures between these three earthquake codes are summarized in Table 2.1.

Table 2.1. Differences in Selecting and Scaling Procedures among the Employed Earthquake Codes

	TEC	EC	ASCE
Minimum # of Records	3	3	3
Type of Records	Real or Artificial	Real or Artificial	Real or Artificial
Zero Period Acceleration	≥ 5% damped elastic spectrum	≥ 5% damped elastic spectrum	No limitation is defined
Minimum Duration of Stationary Part of Records	≥5T_1 and ≥15 sec	≥10 sec	No limitation is defined
Scaling Limitation	The mean of response spectra must be greater than 90% of the code-based response spectrum	The mean of response spectra must be greater than 90% of the code-based response spectrum	The mean of response spectra must be greater than the code-based response spectrum
Scaling Limitation Range	b/w 0.2T and 2.0T	b/w 0.2T and 2.0T	b/w 0.2T and 1.5T
# of Records	<7; use the maxima of results ≥7; use the mean of results	<7; use the maxima of results ≥7; use the mean of results	<7; use the maxima of results ≥7; use the mean of results

According to Kalkan and Chopra (2012), the ASCE/SEI scaling technique is truly insufficient especially for high-rise buildings case. The records chosen are based on earthquake magnitude. Because in such cases, rupture distance and site characteristics are such that their scaled spectral acceleration at the fundamental period of the structure is significantly greater than design spectrum values at these periods. Kalkan and Chopra (2009) investigate the modal pushover-based scaling procedure in their research for selecting and scaling earthquake ground motions for assessing existing structures and design of new structures.

Reyes et. al. (2014) claims that the scaling procedure developed by Kalkan and Chopra is better than ASCE/SEI 7-10 procedure for scaling both horizontal components of the ground motion recordings, since this scaling procedure gives more accurate median demand parameters with respect to target values. Moreover, Azarbakht and Ashtiany (2008) claims that using scaled ground motion records compatible to earthquake design codes is always conservative.

As it is mentioned by Kalkan and Chopra (2010), there is not any specified scaling factor for each record and each structure in ASCE/SEI scaling method, apparently different combinations of scaling factors are used to fit average spectrum of scaled records to design spectrum over the determined period range. This is also valid for TEC and EUROCODE.

CHAPTER 3

PROPERTIES OF BUILDINGS STUDIED

3.1. Introduction

The aim of this chapter is to give information about the buildings used in this study. As it is mentioned before four moment-resisting frame reinforced concrete buildings were modeled with respect to structural member dimensions, material characteristics and soil information obtained from the research named “A Statistical Study on Geometrical Properties of Turkish Reinforced Concrete Building Stock.” (Azak et.al., 2014) to have more realistic building models.

The study by Azak et.al,2014 gives information about geometries of buildings such as floor dimensions, column and beam sizes, minimum and maximum span lengths etc. in Zeytinburnu, Küçükçekmece and Bakırköy districts of İstanbul and in Düzce. Due to the fact that the buildings observed in the study made by Azak et.al. were constructed before Turkish Earthquake Code 2007 took effect and also Kocaeli (August 1999) and Düzce (November 1999) earthquakes happened, some dimensions of structural members do not satisfy the requirements in Turkish Earthquake Code 2007.

Since there is no reinforcement information reviewed in that study (Azak et.al,2014), first the buildings, which are to be analyzed, were modeled in Probina in three dimensions to obtain reinforcement data. Then, one continuous frame from each building was selected and modeled in Seismostruct software as a two-dimensional frame to conduct response spectrum and time history analysis.

3.2. Building Models

Four moment-resisting reinforced concrete frame buildings which are designed by considering typical Turkish residential buildings defined by Azak et.al (2014), are modeled. These buildings have different stories – 3, 4, 6 and 8 floors – to cover a reasonable period band for scaling method. It is assumed that the buildings are constructed in the first earthquake zone. Three-dimensional models are prepared by using Probina.

The properties of these buildings are presented in Table 3.1.

Table 3.1. Characteristics of Buildings

	3 STORY	4 STORY	6 STORY	8 STORY
First Floor Story Height	3.00m	3.00m	3.00m	3.00m
General Story Heights	2.80m	2.80m	2.80m	2.80m
Plan Dimensions	9.75mx15.60 m	11.00mx15.50m	11.50mx16.00m	13.00mx18.00m
Number of Continuous Frames	3 (inside) 4 (outside)	1 (inside) 4 (outside)	1 (inside) 4 (outside)	1 (inside) 4 (outside)
Span Lengths (m)	1.20 3.00 3.60 4.25 5.50	1.00 1.50 3.00 3.50 4.00	1.00 1.75 2.50 2.75 3.00 3.50 4.50	1.35 1.50 2.00 2.65 3.00 4.00 4.15 4.50

Table 3.1. (Cont.) Characteristics of Buildings

Size of Columns (cm)	60x25 40x25 30x50 25x45 45x30 40x30 50x25 35x25	25x50 25x60 25x45 25x55	30x60 30x65 30x70 25x60 25x65	30x75 30x60 25x60
Slab Thickness	14cm	14cm	14cm	14cm
Beam Dimensions (cm)	25x50	25x50	25x50	25x50 25x40 25x30
Dead Load on Slabs	0.45 t/m2	0.45 t/m2	0.45 t/m2	0.45 t/m2
Live Load on Slabs	0.20t/m2	0.20t/m2	0.20t/m2	0.20t/m2
Concrete Class	C20	C20	C25	C25
Reinforcement Steel Class	S420	S420	S420	S420
Soil Type	Z3	Z3	Z3	Z3

Total seismic weight (calculated as the sum of dead weights and %30 live load), first three mode periods and base shear force obtained from Probina models are shown in Table 3.2.

Table 3.2. Dynamic properties of Buildings obtained from Probina Models

	Total Seismic Weight (t)	1st Mode Period (s)	2nd Mode Period (s)	3rd Mode Period (s)	Base Shear (t)
3-Story Building	520.12	0.415	0.373	0.345	65.01
4-Story Building	787.98	0.524	0.496	0.432	98.50
6-Story Building	1309.21	0.64	0.612	0.536	91.78
8-Story Building	2217.78	0.824	0.765	0.669	131.67

One continuous frame for each building is extracted and is modeled as two-dimensional frame in Seismostruct software. Reinforcement details of columns and beams were obtained from the design carried out in Probina. Slab weights, dead and live loads were applied to beams as uniformly distributed loads.

3.2.1. 3-Story Building

Following figure 3.1 shows floor plan and the selected frame of 3-story building.

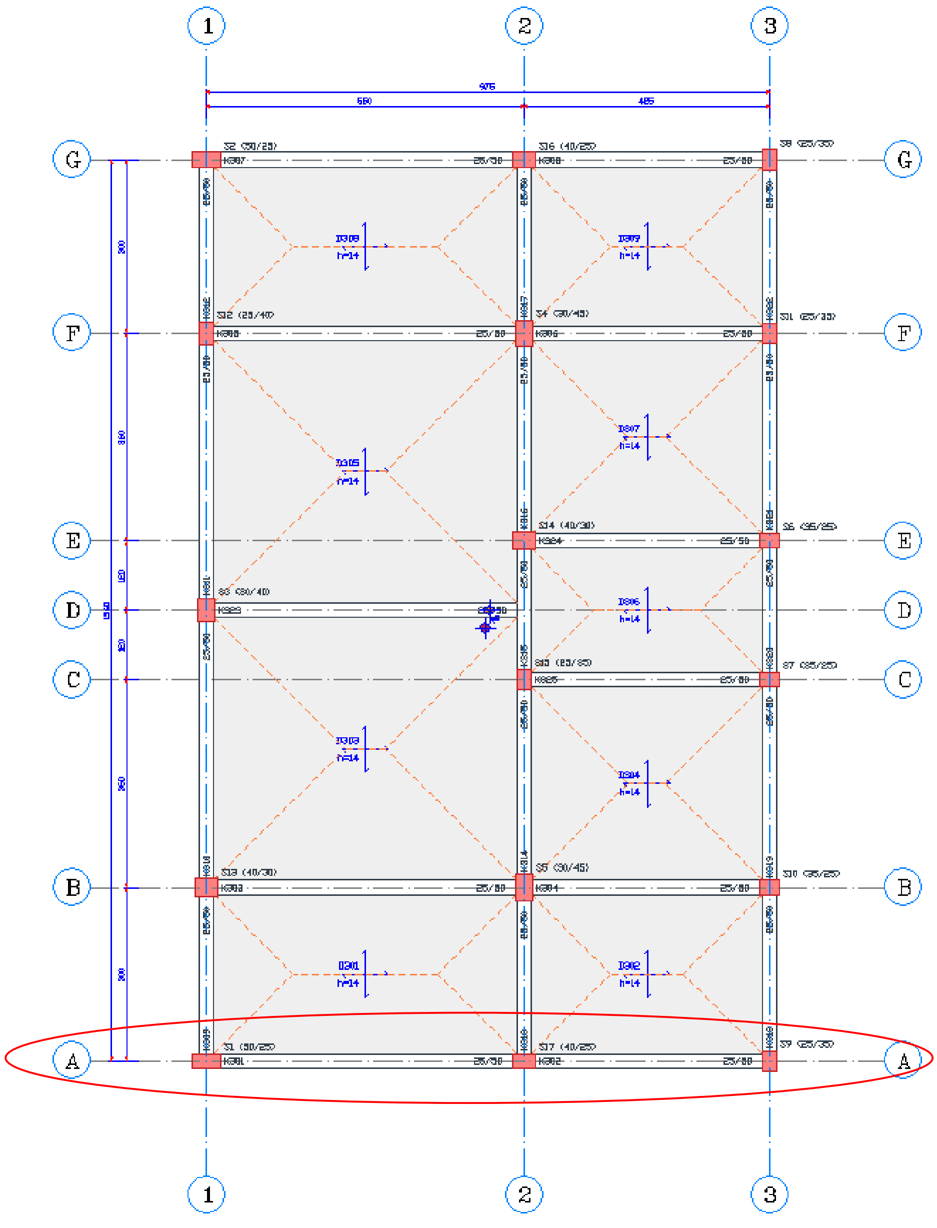

Figure 3.1. Floor Plan and Selected Frame for Analysis of 3-Story Building

The Frame B-B was selected for eigenvalue, static pushover, response spectrum and time history analysis. The reinforcement information obtained from Probina model for Frame B-B is tabulated in Table 3.3. Detailed reinforcement drawings are given in Appendix A.

Table 3.3. Reinforcement Details of 3-story frame

FIRST FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S13	6ϕ18	**K103**	2ϕ12 (top) 5ϕ12 (bottom)	5ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S5	6ϕ18	**K104**	2ϕ12 (top) 5ϕ12 (bottom)	5ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S10	6ϕ16			
SECOND FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S13	6ϕ18	**K203**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S5	6ϕ18	**K204**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S10	6ϕ16			
THIRD FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S13	6ϕ16	**K303**	2ϕ12 (top) 3ϕ12 (bottom)	4ϕ12 (top) 4ϕ12(bottom)
S5	6ϕ18	**K304**	2ϕ12 (top) 3ϕ12 (bottom)	4ϕ12 (top) 4ϕ12(bottom)
S10	6ϕ14			

The Seismostruct model of this frame can be seen in Figure 3.2.

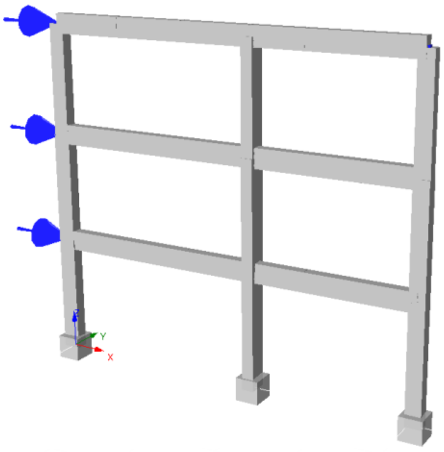

Figure 3.2. Seismostruct Frame Model of 3-Story Building

After eigenvalue analysis of the selected frame was performed, the fundamental period was found as 0.33sec. The total seismic weight, first three periods and modal shapes obtained from eigenvalue analysis are shown in Table 3.4 and Figure 3.3.

Table 3.4. Eigenvalue Analysis Result of 3-Story Frame

	Total Seismic Weight (t)	1st Mode Period (s)	2nd Mode Period (s)	3rd Mode Period (s)
3-Story Frame	45.46	0.33	0.11	0.07

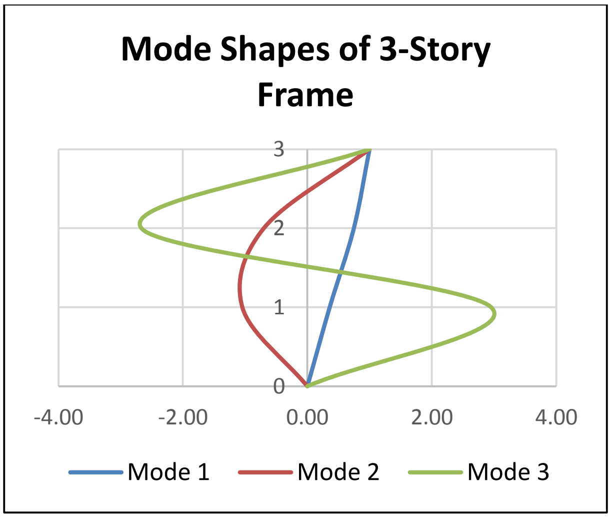

Figure 3.3. Mode Shapes of 3-Story Frame

3.2.2. 4-Story Building

Following figure 3.4 shows floor plan and the selected frame of 4-story building.

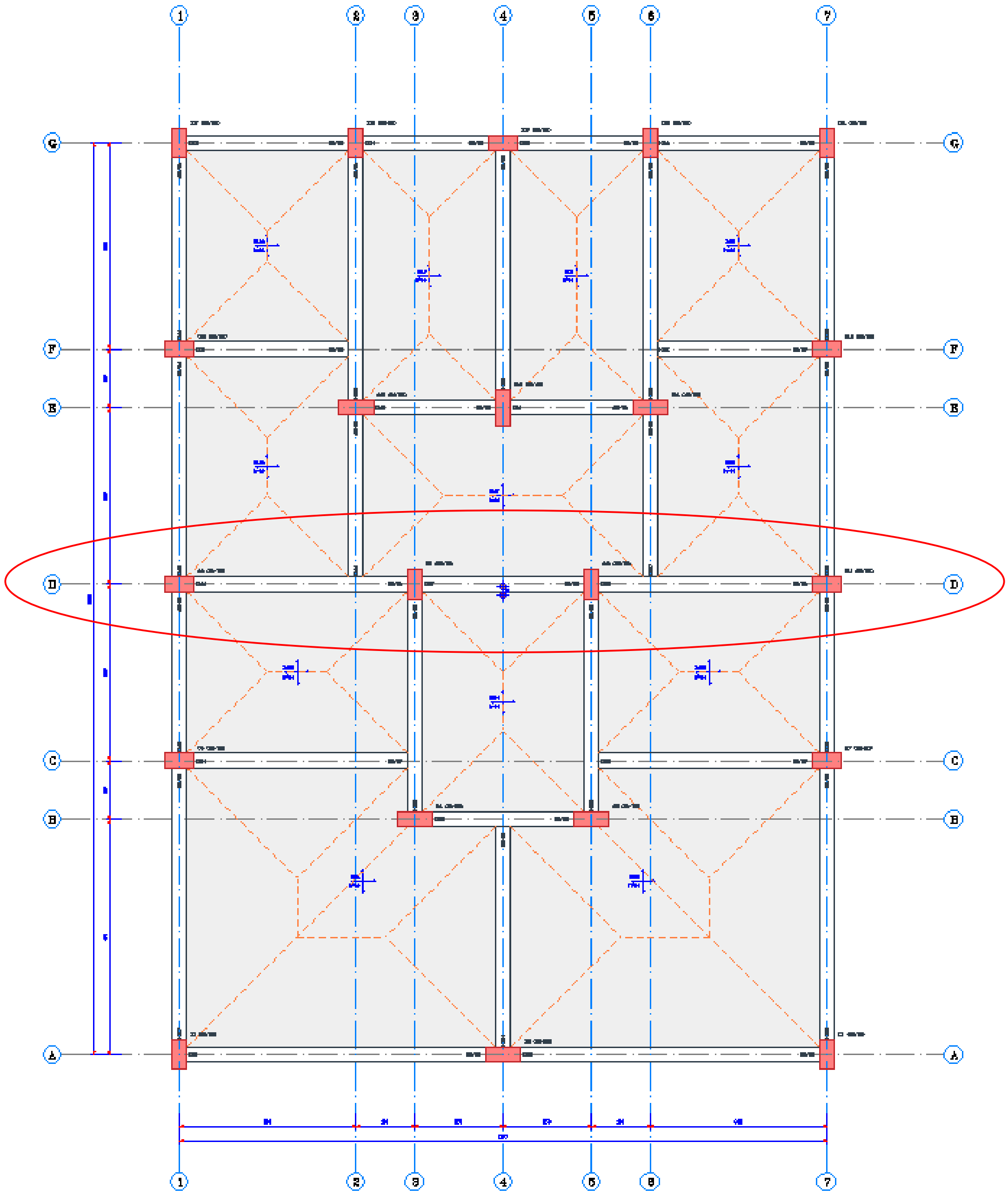

Figure 3.4. Floor Plan and Selected Frame for Analysis of 4-Story Building

Frame D-D was selected for eigenvalue, static pushover, response spectrum and time history analysis. The reinforcement information obtained from Probina model for Frame D-D is shown in Table 3.5. Detailed reinforcement drawings are given in Appendix A.

Table 3.5. Reinforcement Details of 4-story frame

FIRST FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S8	6ϕ18	**K106**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S9	6ϕ18	**K107**	2ϕ12 (top) 4ϕ12 (bottom)	6ϕ12 (top) 3ϕ12(bottom)
S10	6ϕ18	**K108**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S11	6ϕ18			
SECOND FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S8	6ϕ18	**K206**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)
S9	6ϕ18	**K207**	2ϕ12 (top) 4ϕ12 (bottom)	6ϕ12 (top) 3ϕ12(bottom)
S10	6ϕ18	**K208**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12+2ϕ14 (top) 4ϕ12(bottom)

Table 3.5. (Cont.) Reinforcement Details of 4-story frame

S11	**6ϕ18**			
THIRD FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S8	6ϕ16	**K306**	2ϕ12 (top) 4ϕ12 (bottom)	5ϕ12 (top) 3ϕ12(bottom)
S9	6ϕ16	**K307**	2ϕ12 (top) 4ϕ12 (bottom)	6ϕ12 (top) 3ϕ12(bottom)
S10	6ϕ16	**K308**	2ϕ12 (top) 4ϕ12 (bottom)	5ϕ12 (top) 3ϕ12(bottom)
S11	6ϕ16			
FOURTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S8	6ϕ16	**K406**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12 (top) 3ϕ12(bottom)
S9	6ϕ16	**K407**	2ϕ12 (top) 4ϕ12 (bottom)	6ϕ12 (top) 3ϕ12(bottom)
S10	6ϕ16	**K408**	2ϕ12 (top) 4ϕ12 (bottom)	4ϕ12 (top) 3ϕ12(bottom)
S11	6ϕ16			

The Seismostruct model of this frame can be seen in Figure 3.5.

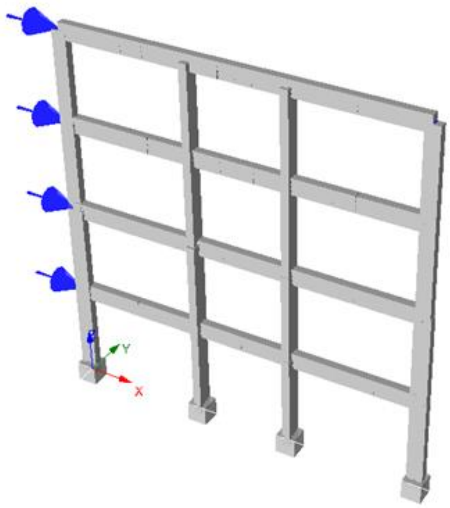

Figure 3.5. Seismostruct Frame Model of 4-Story Building

After eigenvalue analysis of the selected frame was performed, fundamental period is found as 0.40sec. The total seismic weight, first three periods and modal shapes obtained from eigenvalue analysis are shown in Table 3.6 and Figure 3.6.

Table 3.6. Eigenvalue Analysis Result of 4-Story Frame

	Total Seismic Weight (t)	1st Mode Period (s)	2nd Mode Period (s)	3rd Mode Period (s)
4-Story Frame	67.49	0.40	0.14	0.08

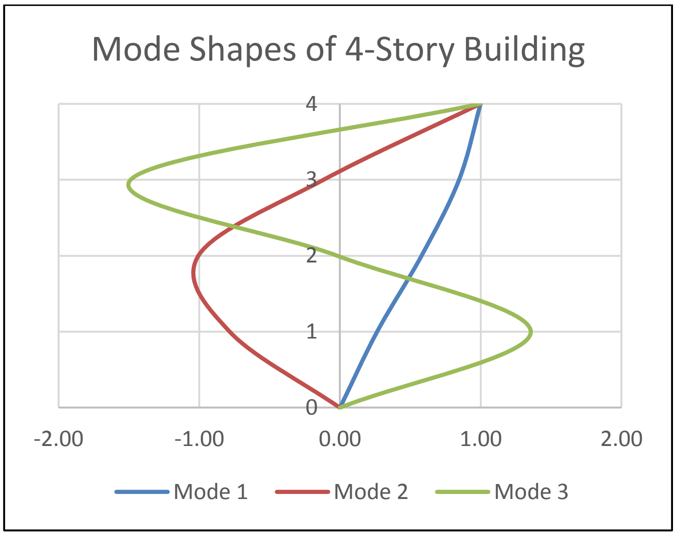

Figure 3.6. Modal Shapes of 4-Story Frame

3.2.3. 6-Story Building

Following figure 3.7 shows floor plan and selected frame of 6-story building for analysis.

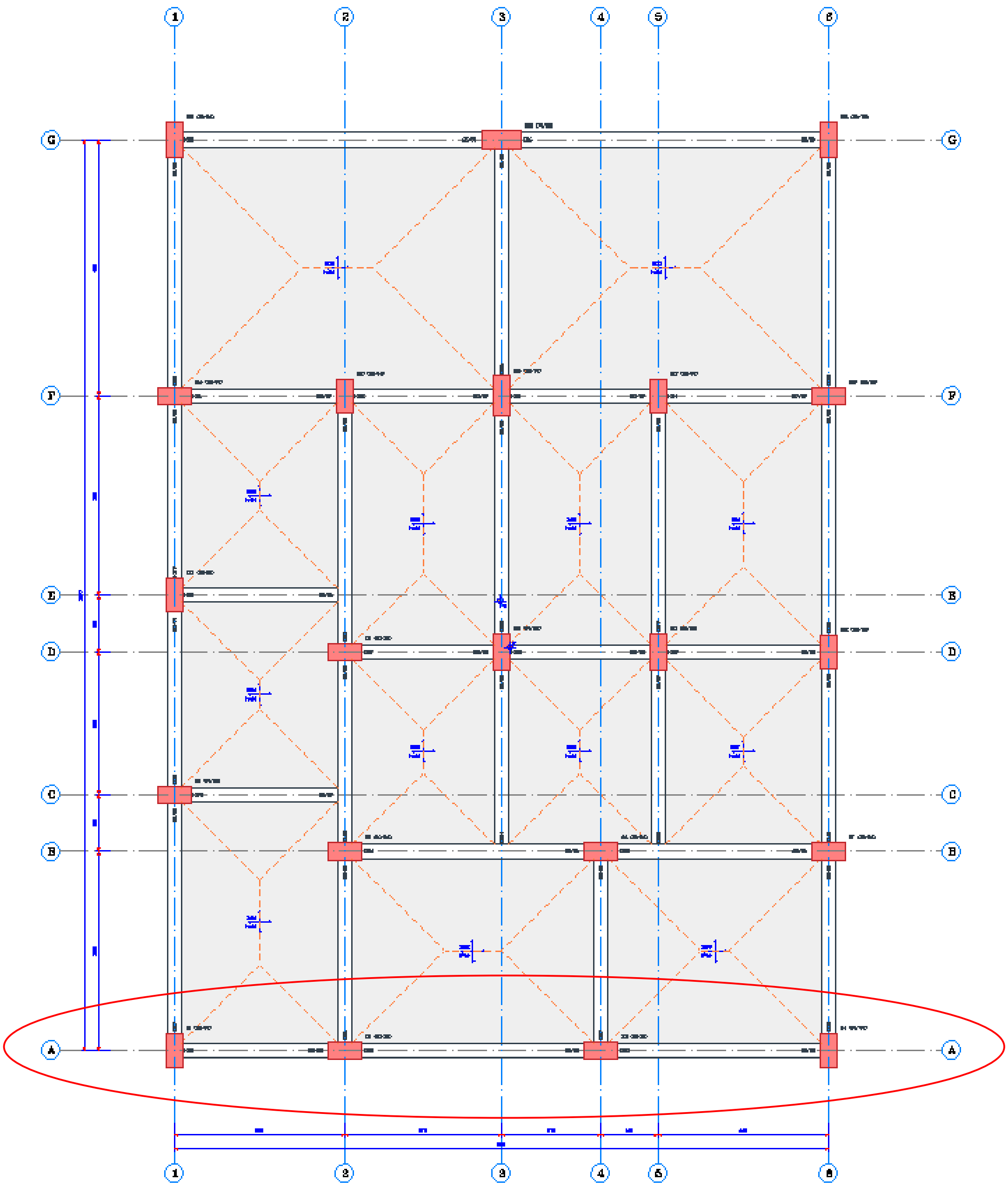

Figure 3.7. Floor Plan and Selected Frame for Analysis of 6-Story Building

Frame A-A was selected for eigenvalue, static pushover, response spectrum and time history analysis. The reinforcement information obtained from Probina model for Frame A-A is tabulated in Table 3.7. Detailed reinforcement drawings are shown in Appendix A.

Table 3.7. Reinforcement Details of 6-story frame

FIRST FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ20	**K101**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S2	8ϕ18	**K102**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S3	8ϕ18	**K103**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S4	8ϕ20			
SECOND FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ18	**K201**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S2	8ϕ18	**K202**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S3	8ϕ18	**K203**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S4	8ϕ18			

Table 3.7. (Cont.)Reinforcement Details of 6-story frame

THIRD FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ18	**K301**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+1ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S2	8ϕ18	**K302**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 2ϕ16+3ϕ12 (bottom)
S3	8ϕ18	**K303**	3ϕ12 (top) 3ϕ12 (bottom)	3ϕ12+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S4	8ϕ18			
FOURTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ16	**K401**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+1ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S2	8ϕ16	**K402**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S3	8ϕ16	**K403**	3ϕ12 (top) 3ϕ12 (bottom)	2ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S4	8ϕ16			
FIFTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ16	**K501**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S2	8ϕ16	**K502**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+1ϕ20 (top) 5ϕ12 (bottom)
S3	8ϕ16	**K503**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S4	8ϕ16			

Table 3.7. (Cont.)Reinforcement Details of 6-story frame

SIXTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S1	8ϕ16	**K601**	2ϕ14 (top) 3ϕ12 (bottom)	2ϕ14 (top) 3ϕ12(bottom)
S2	8ϕ16	**K602**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12 (bottom)
S3	8ϕ16	**K603**	2ϕ14 (top) 3ϕ12 (bottom)	2ϕ14 (top) 3ϕ12(bottom)
S4	8ϕ16			

Modeled frame A-A in Seismostruct is shown in Figure 3.8.

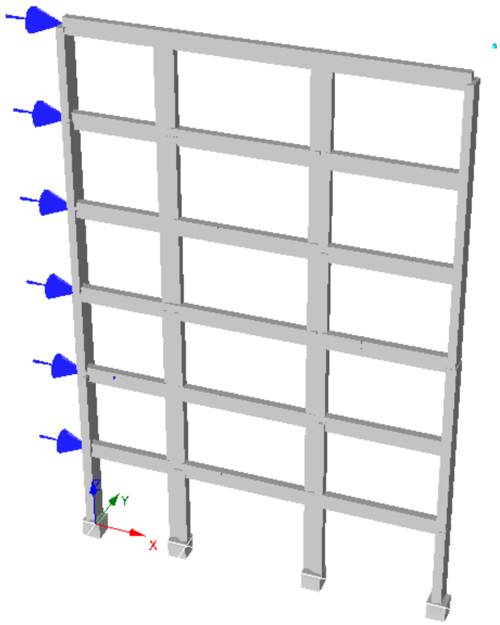

Figure 3.8. Seismostruct Frame Model of 6-Story Building

After eigenvalue analysis was finished, fundamental period is found as 0.53sec. The total seismic weight, first three periods and modal shapes obtained from eigenvalue analysis are shown in Table 3.8 and Figure 3.9.

Table 3.8. Eigenvalue Analysis Result of 6-Story Frame

	Total Seismic Weight (t)	1st Mode Period (s)	2nd Mode Period (s)	3rd Mode Period (s)
6-Story Frame	148.14	0.53	0.18	0.10

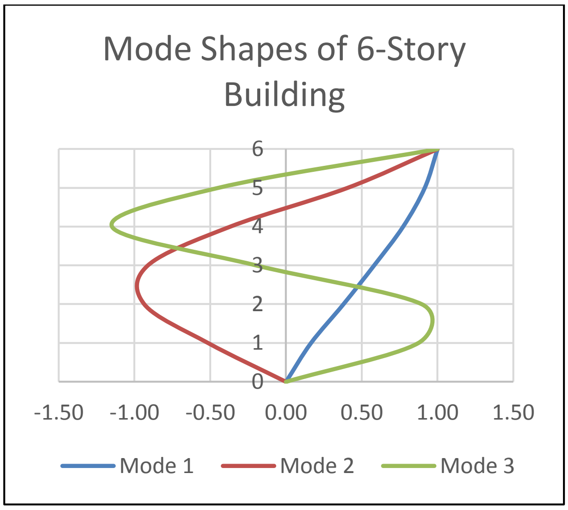

Figure 3.9. Modal Shapes of 6-Story Frame

3.2.4. 8-Story Building

Figure 3.10 shows floor plan and selected frame of 8-story building for analysis.

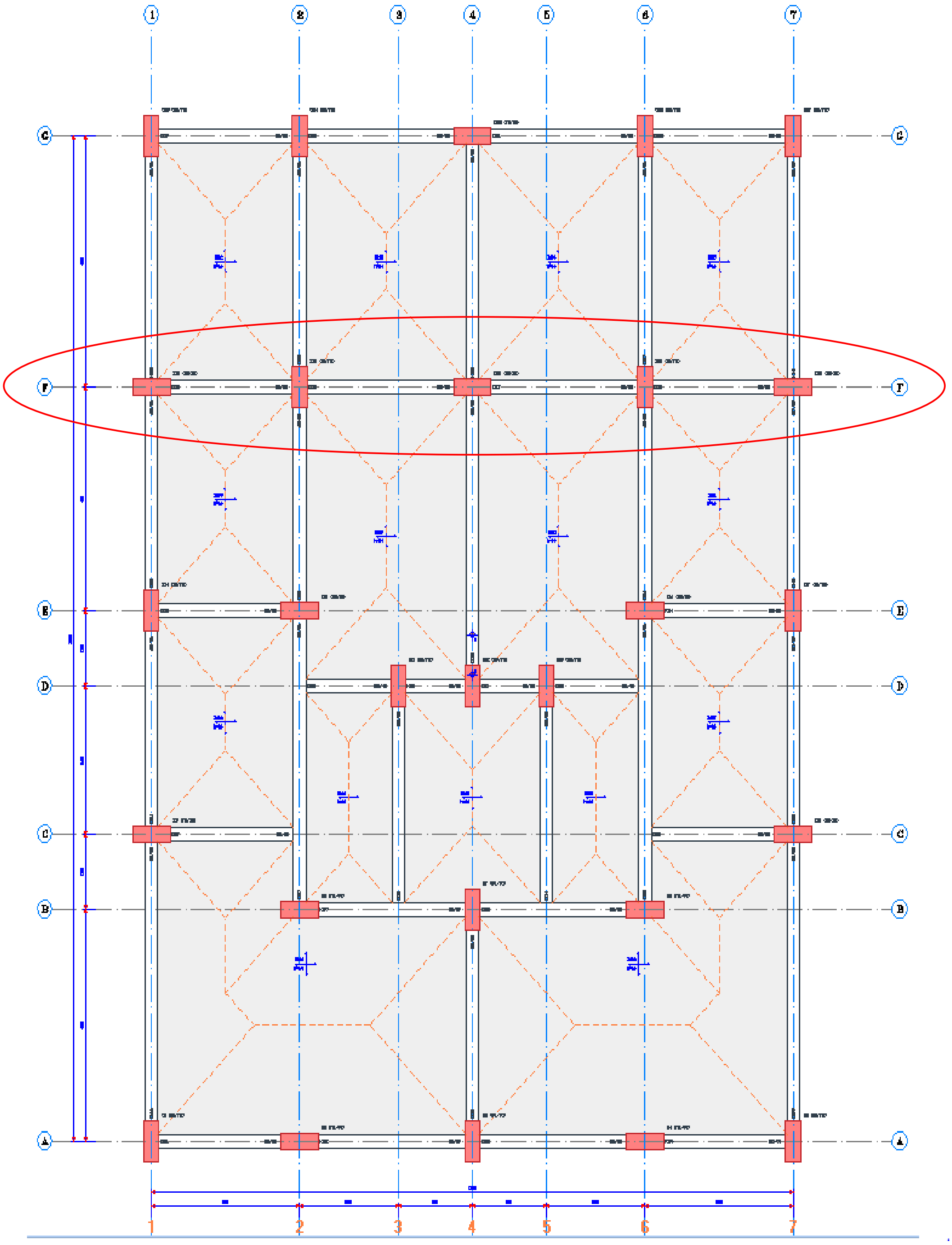

Figure 3.10. Floor Plan and Selected Frame for Analysis of 8-Story Building

Frame F-F was selected for eigenvalue, static pushover and time history analysis. The reinforcement information obtained from Probina model for Frame A-A can be seen in Table 3.9. Detailed reinforcement drawings are given in Appendix A.

Table 3.9. Reinforcement Details of 8-story frame

FIRST FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	10ϕ20	**K115**	3ϕ16 (top) 3ϕ12+2ϕ20 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ20+3ϕ12 (bottom)
S19	10ϕ18	**K116**	3ϕ14 (top) 3ϕ12+2ϕ18 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S20	10ϕ18	**K117**	3ϕ14 (top) 3ϕ12+2ϕ18 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S21	10ϕ18	**K118**	3ϕ16 (top) 3ϕ12+2ϕ20 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ20+3ϕ12 (bottom)
S22	10ϕ20			
SECOND FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	10ϕ18	**K215**	3ϕ16 (top) 3ϕ12+2ϕ20 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ20+3ϕ12 (bottom)
S19	10ϕ18	**K216**	3ϕ14 (top) 3ϕ12+2ϕ18 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S20	10ϕ18	**K217**	3ϕ14 (top) 3ϕ12+2ϕ18 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S21	10ϕ18	**K218**	3ϕ16 (top) 3ϕ12+2ϕ20 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ20+3ϕ12 (bottom)
S22	10ϕ18			

Table 3.9. (Cont.) Reinforcement Details of 8-story frame

THIRD FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	10ϕ18	**K315**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S19	10ϕ18	**K316**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ16+3ϕ12 (bottom)
S20	10ϕ18	**K317**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ16+3ϕ12 (bottom)
S21	10ϕ18	**K318**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ18+3ϕ12 (bottom)
S22	10ϕ18			
FOURTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	8ϕ18	**K415**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ16+3ϕ12 (bottom)
S19	8ϕ18	**K416**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S20	8ϕ18	**K417**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ14+3ϕ12 (bottom)
S21	8ϕ18	**K418**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 2ϕ16+3ϕ12 (bottom)
S22	8ϕ18			
FIFTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	8ϕ18	**K515**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 5ϕ12 (bottom)
S19	8ϕ18	**K516**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 5ϕ12 (bottom)
S20	8ϕ18	**K517**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 5ϕ12 (bottom)
S21	8ϕ18	**K518**	3ϕ16 (top) 3ϕ12 (bottom)	3ϕ16+2ϕ20 (top) 5ϕ12 (bottom)
S22	8ϕ18			

Table 3.9. (Cont.) Reinforcement Details of 8-story frame

SIXTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	8ϕ18	**K615**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S19	8ϕ18	**K616**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S20	8ϕ18	**K617**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S21	8ϕ18	**K618**	3ϕ14 (top) 3ϕ12 (bottom)	3ϕ14+2ϕ20 (top) 5ϕ12 (bottom)
S22	8ϕ18			
SEVENH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	8ϕ16	**K715**	3ϕ12 (top) 3ϕ12 (bottom)	3ϕ12+1ϕ20 (top) 3ϕ12 (bottom)
S19	8ϕ16	**K716**	3ϕ12 (top) 3ϕ12 (bottom)	2ϕ14+1ϕ18+1ϕ12 (top) 3ϕ12 (bottom)
S20	8ϕ16	**K717**	3ϕ12 (top) 3ϕ12 (bottom)	2ϕ14+1ϕ18+1ϕ12 (top) 3ϕ12 (bottom)
S21	8ϕ16	**K718**	3ϕ12 (top) 3ϕ12 (bottom)	3ϕ12+1ϕ20 (top) 3ϕ12 (bottom)
S22	8ϕ16			
EIGHTH FLOOR				
Columns	**Reinforcement**	**Beams**	**Reinforcement (Mid Span)**	**Reinforcement (Span Ends)**
S18	8ϕ16	**K815**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S19	8ϕ16	**K816**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S20	8ϕ16	**K817**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S21	8ϕ16	**K818**	2ϕ14 (top) 3ϕ12 (bottom)	3ϕ14 (top) 3ϕ12(bottom)
S22	8ϕ16			

The Seismostruct model of Frame F-F can be seen in Figure 3.11.

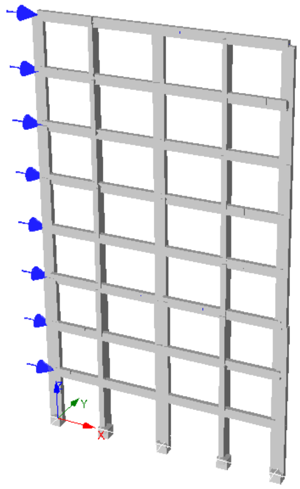

Figure 3.11. Seismostruct Frame Model of 8-Story Building

The fundamental period of the frmae was found as 0.78sec. The total seismic weight, first three periods and modal shapes obtained from eigenvalue analysis are shown in Table 3.10 and Figure 3.12.

Table 3.10. Eigenvalue Analysis Result of 8-Story Frame

	Total Seismic Weight (t)	1st Mode Period (s)	2nd Mode Period (s)	3rd Mode Period (s)
8-Story Frame	343.86	0.78	0.27	0.15

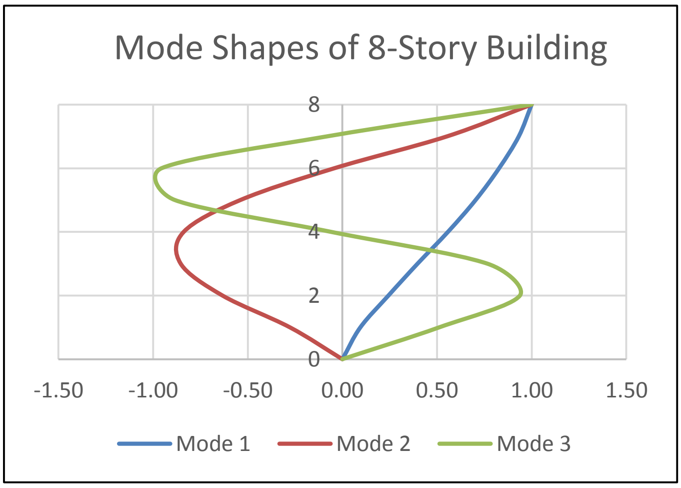

Figure 3.12. Modal Shapes of 8-Story Frame

3.3. Pushover Analysis of Selected Frames

The pushover analysis of the frames were conducted to obtain the deformation limits of each frame.

First, force distribution factors applied to each floor for static pushover analysis are determined using Eq3.1 defined in FEMA 356.

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{Eq.3.1}$$

where;

- C_{vx} is the vertical distribution factor
- k equals to 1.0 for fundamental periods less than 0.5 seconds and interpolates between 1.0 and 2.0 for fundamental period range between 0.5 seconds and 2.5 seconds.
- w_i, w_x are the partial/individual floor weight at i^{th} or x^{th} floor.
- h_i, h_x are the height of the floor between the base and i^{th} or x^{th} floor.

The pushover curve is also idealized based on equal area principle specified by FEMA 356. According to FEMA 356, after nonlinear relationship between base shear and roof displacement of the frame is found from pushover analysis, the effective lateral stiffness and the effective yield strength were calculated to idealize the pushover curve. The effective lateral stiffness is the secant stiffness obtained from a base shear force equal to 60% of the effective yield strength of the structure. Post yield slope is determined by a line, which crosses the real pushover curve at target displacement. Typical idealized pushover curve with respect to FEMA 356 is shown in Figure 3.13.

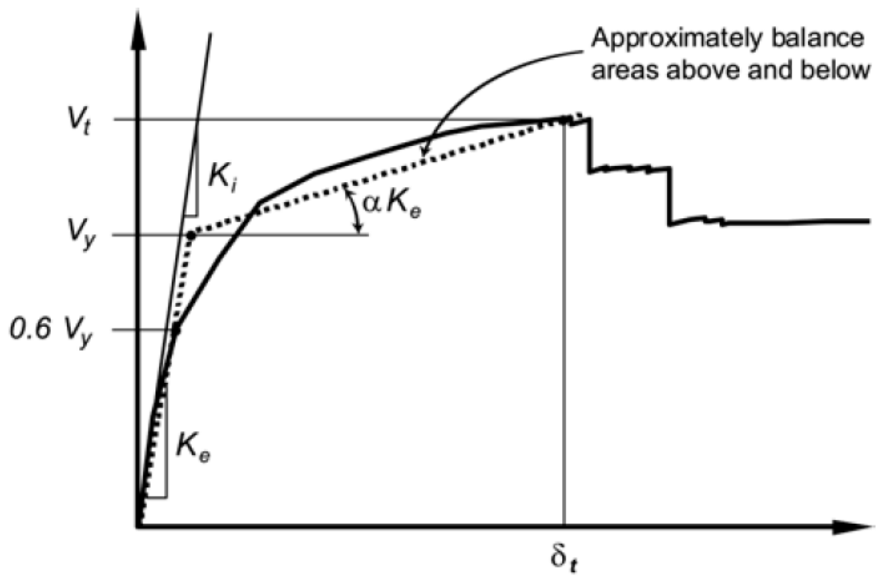

Figure 3.13. Idealized Pushover Curve according to FEMA 356

After the pushover curve is idealized, by using elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis (T_i), elastic lateral stiffness of the building in the direction under consideration (K_i) and effective lateral stiffness of the building in the direction under consideration (K_e), the effective fundamental period in the direction (T_e) under consideration are calculated from Equation 3.2.

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \qquad \text{Eq.3.2}$$

According to the fundamental period obtained from eigenvalue analysis, load patterns that are used in pushover analysis are determined.

As a result of pushover analysis, pushover curve, yield force, ultimate force, spectral displacement and pseudo spectral acceleration were determined. The results of pushover analyses for all frames are summarized in Table 3.12 and Figure 3.14.

Table 3.12. Pushover Analysis Results of Frames

	3-Story	4-Story	6-Story	8-Story
Yield Displacement (m)	0.053	0.064	0.090	0.096
Ultimate Displacement (m)	0.300	0.350	0.485	0.598
Yield Base Shear (kN)	185.000	210000	415.000	550.000
Ultimate Base Shear (kN)	202.686	232036	423.280	569.198
Sd_{yield} (cm)	2.909	3429	6.884	7.145
$Sd_{ultimate}$ (cm)	16.511	18858	37.008	44.282
PSa_{yield} (cm/s^2)	466.912	365668	342.471	206.051
$PSa_{ultimate}$ (cm/s^2)	511.549	404039	349.304	213.243
Effective Fundamental Period (s)	0.50	0.61	0.79	1.12
K_i (kN/m)	8034.89	7674.56	10220.22	11752.3
K_e (kN/m)	3500	3300	4600	5700

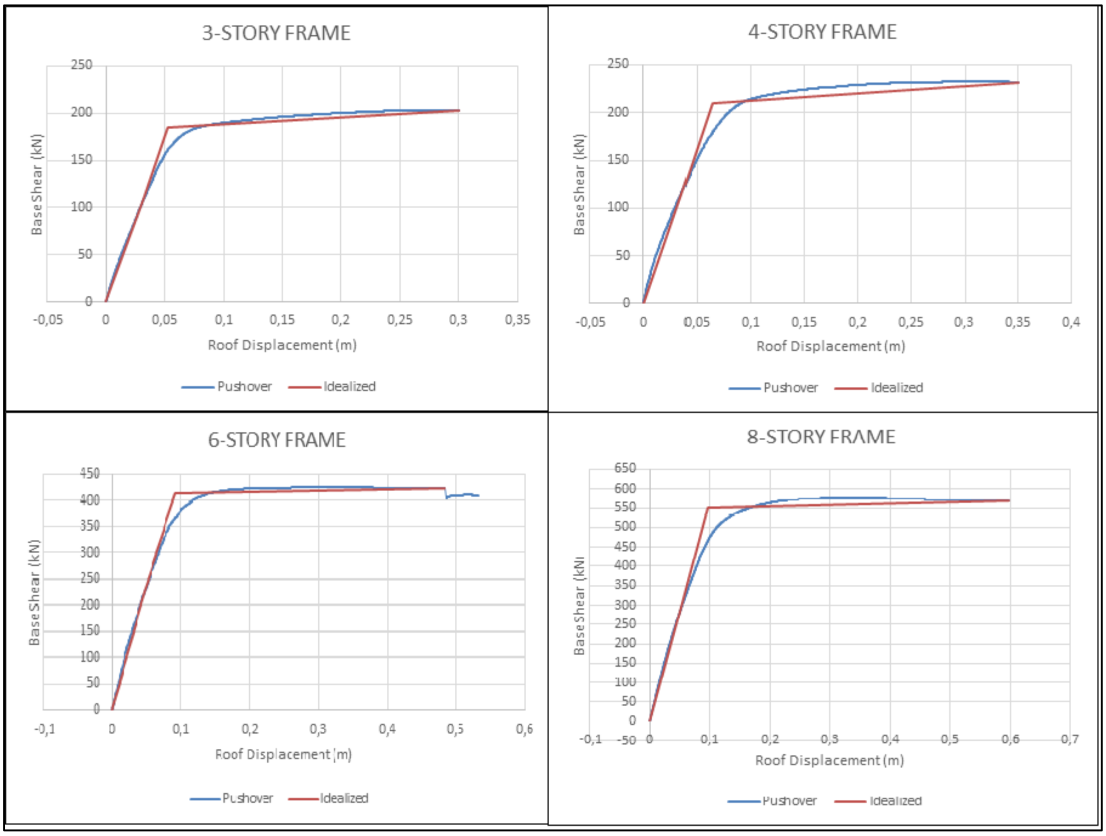

Figure 3.14. Pushover Curves Of Frames

CHAPTER 4

SCALING OF GROUND MOTIONS AND SEISMIC ANALYSES OF BUILDINGS

4.1. Introduction

Ground motion records used in design or analysis of structures shall satisfy the conditions defined in earthquake codes. Therefore, proper selection and scaling of ground motions must be done to empower response history analysis as it is mentioned in the research named Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses made by NEHRP Consultants Joint Venture (2011).

In this chapter of the study, ground motion record sets were selected using the criteria presented in Section 4.2. The selected ground motion records were then scaled to be compatible with the employed code elastic response spectra.

As mentioned earlier three different response spectra were used. A total of fifty one ground motion records were selected for scaling. According to fault mechanisms, five scaling sets were formed. Two different methods that are explained in detail in Section 4.4 were utilized for scaling of ground motions.

Linear dynamic analyses of all four frames were carried out using the code-defined response spectra as well as the scaled ground motion set that is considered to be the most compatible one with the code elastic response spectra. The results of these analyses are compared in this chapter.

4.2. Code Based Response Spectra

Before the scaling was made, response spectra with respect to all three earthquake codes were constructed by selecting the earthquake zone as one and soil type as C (Z3).

These benchmarks were selected to have scaling data that are consistent with the building and frame modeling.

4.2.1. Response Spectra in TEC 2007

Design acceleration spectrum ($S_{ae}(T)$) is the 5% damped elastic spectral acceleration defined by Equation 4.1 in Turkish Earthquake Code 2007.

$$S_{ae}(T) = A(T)g \qquad \text{Eq.4.1}$$

A(T), given in Eq.4.2, is the spectral acceleration coefficient used for determination of seismic loads in analysis.

$$A(T) = A_0 \, I \, S(T) \qquad \text{Eq.4.2}$$

where:

- A_0 is the effective ground acceleration coefficient and is given as 0.40 for first seismic zone.
- I is the building importance factor and is specified as 1.0 for residential and office buildings, hotels etc.
- S(T), which is defined by Eq.4.3, is the spectrum coefficient that depends on site conditions and the natural period of the building.
-

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad (0 \le T \le T_A)$$

$$S(T) = 2.5 \qquad (T_A \le T \le T_B) \qquad \text{Eq.4.3}$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B < T)$$

- As the local site class is assumed as Z3 for the study, spectrum characteristic periods T_A and T_B are taken as 0.15 and 0.60 seconds, respectively.

TEC2007 code-based response spectrum based on the parameters given above is shown in Figure 4.1.

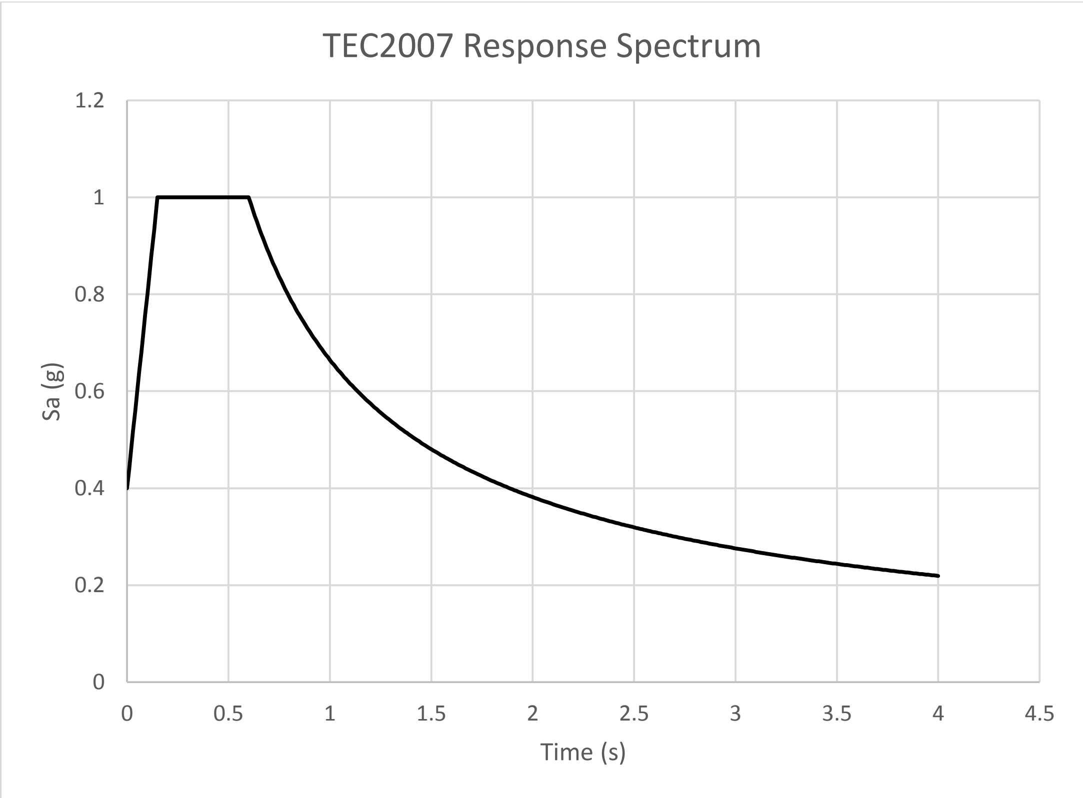

Figure 4.1. TEC2007 Response Spectrum

4.2.2. Response Spectra in Eurocode 8

There are both horizontal and vertical elastic response spectra defined in Eurocode 8. However, only horizontal elastic response spectrum that is defined by Eq.4.4 was used.

$$S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right] \qquad 0 \leq T \leq T_B$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \qquad T_B \leq T \leq T_C \qquad \text{Eq.4.4}$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C}{T}\right] \qquad T_C \leq T \leq T_D$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C T_D}{T}\right] \qquad T_D \leq T \leq 4$$

where:

- a_g is the design ground acceleration on type A ground and is defined as 0.4 in Eurocode 8.
- S is the soil factor and for C (Z3) type soil is equal to 1.15.
- T_B and T_C are the lower and upper period limits of the constant spectral acceleration branch and are taken as 0.20 and 0.60 seconds, respectively.
- T_D is the beginning period of constant displacement range and is 2.00 seconds for C type soil condition.
- η is the damping correction factor equals to 1 for 5% viscous damping.

According to the statements defined in Eurocode and mentioned above, Eurocode 8 Type 1 elastic response spectrum is calculated and shown in Figure 4.2.

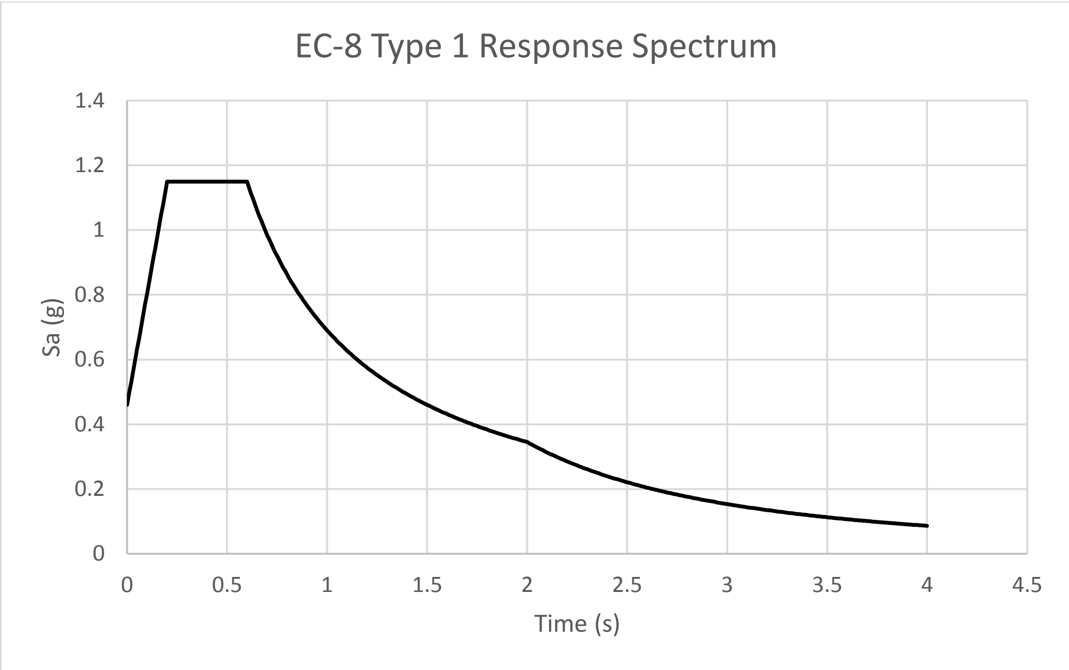

Figure 4.2. EC-8 Response Spectrum

4.2.3. Response Spectra in ASCE 7-10

Design spectral acceleration is constructed regarding Eq.4.5 in ASCE 7-10.

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \qquad T < T_0$$

$$S_a = S_{DS} \qquad T_0 \leq T \leq T_S \qquad \text{Eq.4.5}$$

$$S_a = \frac{S_{D1}}{T} \qquad T_S < T \leq T_L$$

$$S_a = \frac{S_{D1}T_L}{T^2} \qquad T_L < T$$

where:

- S_{DS} is the design spectral response acceleration parameter at short periods calculated using Eq.4.6.

$$S_{DS} = \frac{2}{3}S_{MS}\,, \qquad S_{MS} = F_a S_S \qquad \text{Eq.4.6}$$

- S_S is the mapped maximum considerable earthquake response acceleration at short periods and is equal to 1.53 for this study. This value is provided from the U.S. Geological Survey website (http://geohazards.usgs.gov/designmaps/ww/) for the location of İstanbul, Turkey.
  - F_a is the short-period site coefficient and is equal to 1.
- S_{D1} is the design spectral response acceleration parameter at 1-s period obtained by using Eq.4.7.

$$S_{D1} = \frac{2}{3} S_{M1}\,, \qquad S_{M1} = F_v S_1 \qquad \text{Eq.4.7}$$

  - S_1 is the mapped maximum considerable earthquake response acceleration at a period of 1 second and equals to 0.72 for this study. This value is provided from the U.S. Geological Survey website (http://geohazards.usgs.gov/designmaps/ww/) for the location of İstanbul, Turkey.
    - F_v is the long-period site coefficient equals to 1.30.
- T_0 is equal to 20% of S_{D1}/S_{DS} and found as 0.12 seconds
- T_S is the ratio between S_{D1} and S_{DS} and equals to 0.61 seconds for the study.

ASCE7-10 elastic response spectrum is created by using these formulations and is presented in Figure 4.3.

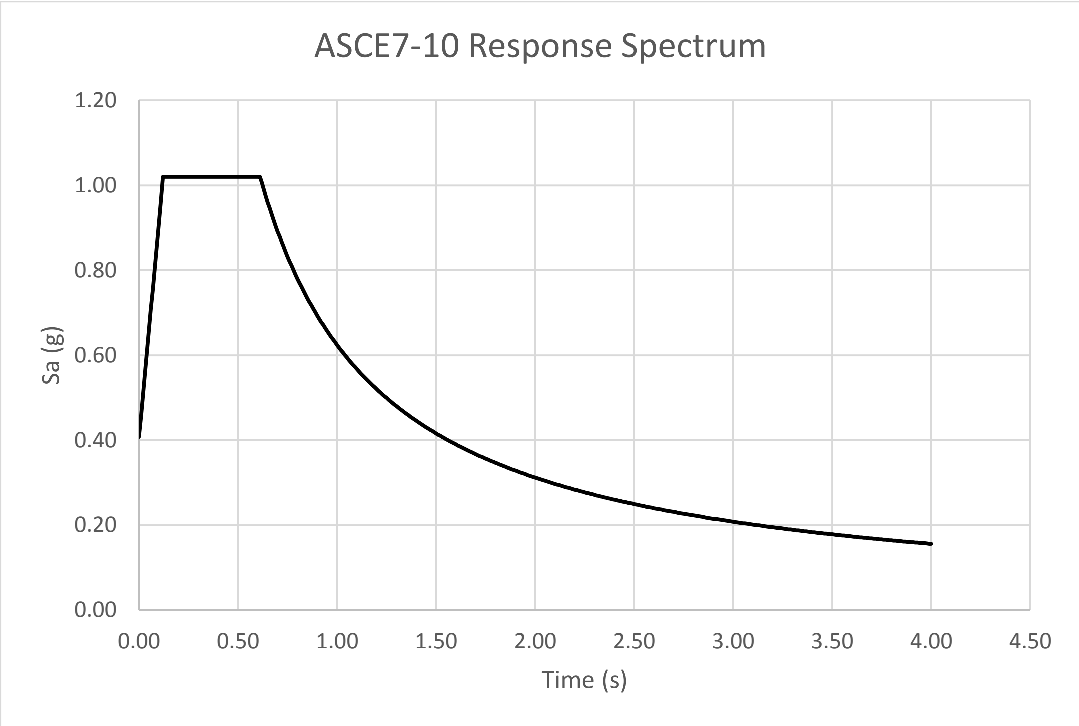

Figure 4.3. ASCE/SEI 7-10 Response Spectrum

4.3. Ground Motion Records

Ground motion records were selected from PEER ground motion database according to the criteria defined below.

- Moment magnitude range of ground motion records was taken between 6.00 and 7.50.
- The Joyner-Boore distance of the ground motion record was between 15 and 30km.
- Soil type was assumed as C (Z3).

51 candidate ground motions from 15 earthquakes were obtained using the criteria defined above. The properties of these ground motions are summarized in Table 4.1.

Table 4.1. Candidate Ground Motion Recordings

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)
28	"Parkfield"	1966	6.19	SS	17.64	408.93	C	0.090
33	"Parkfield"	1966	6.19	SS	15.96	527.92	C	0.456
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430
63	"San Fernando"	1971	6.61	R	25.58	634.33	C	0.135
70	"San Fernando"	1971	6.61	R	22.23	425.34	C	0.190
72	"San Fernando"	1971	6.61	R	19.45	600.06	C	0.254
73	"San Fernando"	1971	6.61	R	17.22	670.84	C	0.226
78	"San Fernando"	1971	6.61	R	24.16	452.86	C	0.191
79	"San Fernando"	1971	6.61	R	25.47	415.13	C	0.149
88	"San Fernando"	1971	6.61	R	24.69	389	C	0.220
164	"Imperial Valley-06"	1979	6.53	SS	15.19	471.53	C	0.232
286	"Irpinia Italy-01"	1980	6.9	N	17.51	496.46	C	0.126
288	"Irpinia Italy-01"	1980	6.9	N	22.54	561.04	C	0.287
290	"Irpinia Italy-01"	1980	6.9	N	29.79	428.57	C	0.177
291	"Irpinia Italy-01"	1980	6.9	N	27.49	574.88	C	0.139
295	"Irpinia Italy-02"	1980	6.2	N	28.69	476.62	C	0.033
296	"Irpinia Italy-02"	1980	6.2	N	17.79	649.67	C	0.075
302	"Irpinia Italy-02"	1980	6.2	N	22.68	574.88	C	0.139
303	"Irpinia Italy-02"	1980	6.2	N	20.38	382	C	0.104
336	"Coalinga-01"	1983	6.36	R	27.1	541.73	C	0.118
340	"Coalinga-01"	1983	6.36	R	26.2	384.26	C	0.234
351	"Coalinga-01"	1983	6.36	R	28.72	450.61	C	0.120
359	"Coalinga-01"	1983	6.36	R	24.83	381.27	C	0.294
362	"Coalinga-01"	1983	6.36	R	29.01	438.74	C	0.120
369	"Coalinga-01"	1983	6.36	R	25.98	648.09	C	0.223
450	"Morgan Hill"	1984	6.19	SS	23.23	462.24	C	0.137
516	"N. Palm Springs"	1986	6.06	RO	27.21	425.17	C	0.227
521	"N. Palm Springs"	1986	6.06	RO	29.56	407.61	C	0.297
524	"N. Palm Springs"	1986	6.06	RO	23.2	379.32	C	0.084
534	"N. Palm Springs"	1986	6.06	RO	22.96	447.22	C	0.351
537	"N. Palm Springs"	1986	6.06	RO	16.55	659.09	C	0.187

Table 4.1. (Cont.) Candidate Ground Motion Recordings

548	"Chalfant Valley-02"	1986	6.19	SS	21.55	370.94	C	0.276
551	"Chalfant Valley-02"	1986	6.19	SS	29.35	382.12	C	0.094
552	"Chalfant Valley-02"	1986	6.19	SS	22.08	456.83	C	0.188
553	"Chalfant Valley-02"	1986	6.19	SS	18.3	537.16	C	0.111
554	"Chalfant Valley-02"	1986	6.19	SS	18.3	537.16	C	0.111
587	"New Zealand-02"	1987	6.6	N	16.09	551.3	C	0.375
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343
740	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.101
755	"Loma Prieta"	1989	6.93	RO	19.97	561.43	C	0.509
769	"Loma Prieta"	1989	6.93	RO	17.92	663.31	C	0.212
775	"Loma Prieta"	1989	6.93	RO	29.54	621.2	C	0.070
815	"Griva Greece"	1990	6.1	N	26.75	454.56	C	0.067
827	"Cape Mendocino"	1992	7.01	R	15.97	457.06	C	0.164
830	"Cape Mendocino"	1992	7.01	R	26.51	518.98	C	0.303
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277
954	"Northridge-01"	1994	6.69	R	19.1	550.11	C	0.309
957	"Northridge-01"	1994	6.69	R	15.87	581.93	C	0.197
963	"Northridge-01"	1994	6.69	R	20.11	450.28	C	0.772
974	"Northridge-01"	1994	6.69	R	21.64	371.07	C	0.430
991	"Northridge-01"	1994	6.69	R	28.98	366.71	C	0.269
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

Spectral acceleration of candidate ground motions and response spectra with respect to earthquake codes are presented in Figure 4.4.

As it can be seen in Figure 4.4 that most of the candidate ground motions have low spectral accelerations and the mean peak ground acceleration of these ground motions is approximately 0.2g.

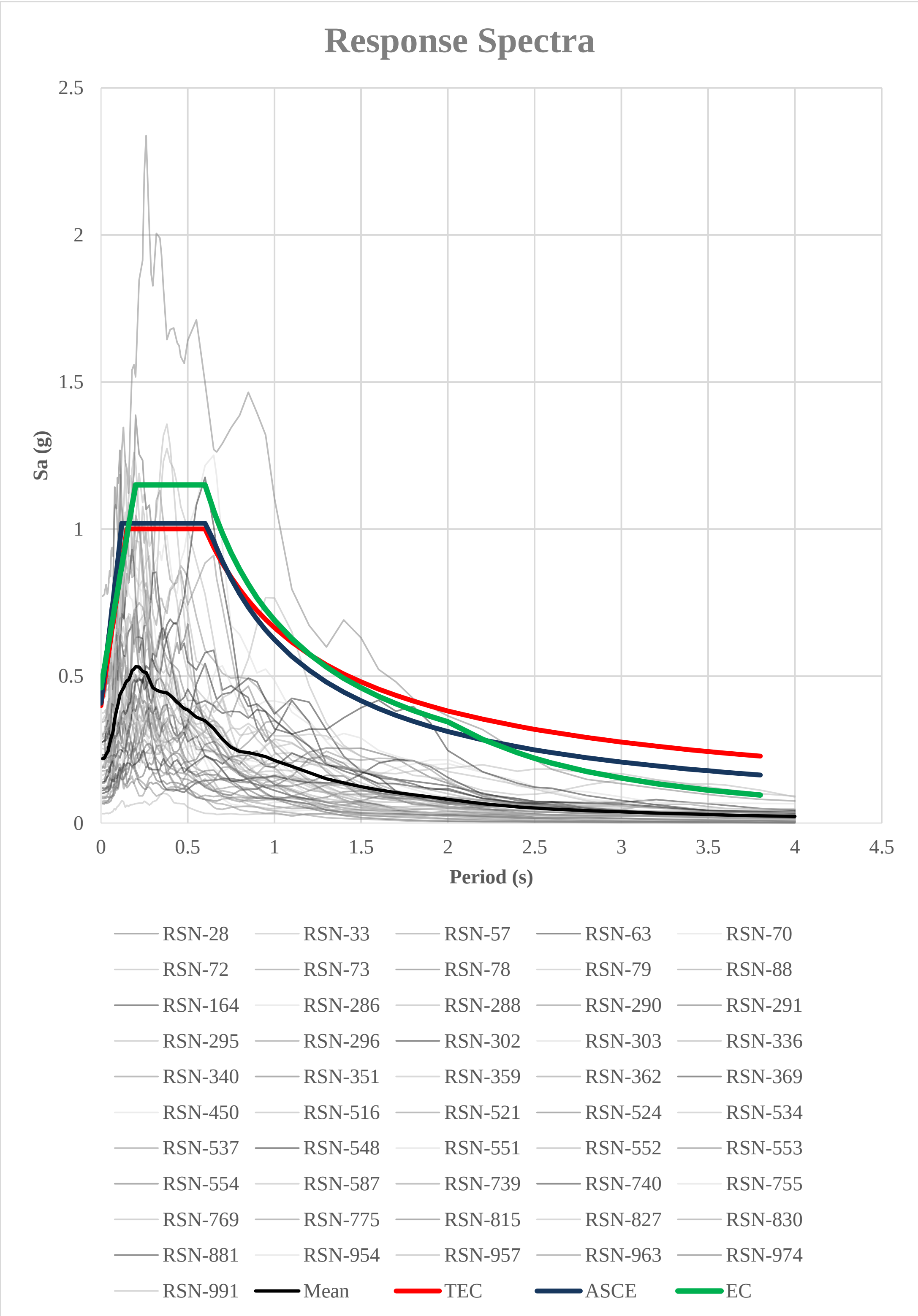

Figure 4.4. Spectral Acceleration of Candidate Ground Motions

Five alternative ground motion sets were generated by using these candidate ground motions. Four of them were formed with respect to the fault mechanism and the other one was generated by eliminating the ground motions, which have too high and too low spectral accelerations.

- Ground Motion Set 1

  This is the ground motion set which is constructed by disregarding the ground motions with outlier spectral accelerations when compared to the code-based elastic response spectra. The properties of the ground motions and spectral accelerations are shown in Table 4.2 and Figure 4.5.

  This set consists of ten ground motion records from ten different earthquakes. The peak ground motion acceleration of the set is in a range between 0.2g and 0.5g.

Table 4.2. Ground Motion Set 1

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)
33	"Parkfield"	1966	6.19	SS	15.96	527.92	C	0.456
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430
164	"Imperial Valley-06"	1979	6.53	SS	15.19	471.53	C	0.232
288	"Irpinia Italy-01"	1980	6.9	N	22.54	561.04	C	0.287
359	"Coalinga-01"	1983	6.36	R	24.83	381.27	C	0.294
548	"Chalfant Valley-02"	1986	6.19	SS	21.55	370.94	C	0.276
587	"New Zealand-02"	1987	6.6	N	16.09	551.3	C	0.375
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277
991	"Northridge-01"	1994	6.69	R	28.98	366.71	C	0.269
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

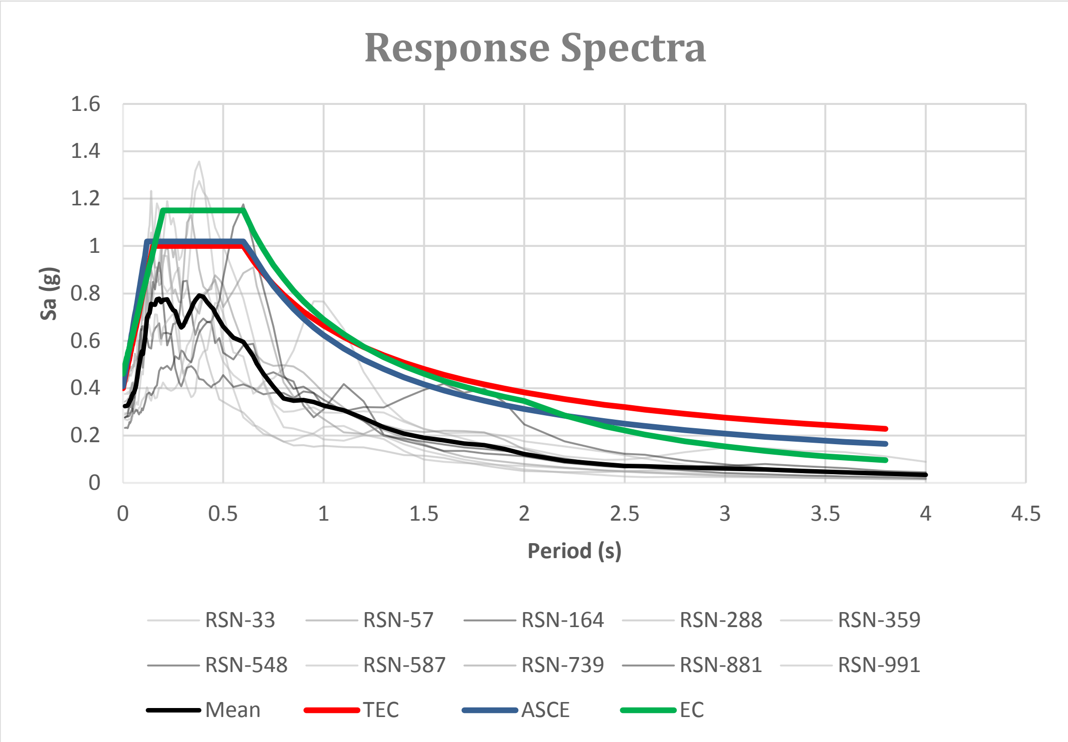

Figure 4.5. Response Spectra of Ground Motion Set 1

- Ground Motion Set 2

  This ground motion set contains only ground motions with strike slip fault mechanism. These ground motions were obtained from 5 different earthquakes. Peak ground accelerations are between 0.09g and 0.3g and in average equals to 0.2g. The ground motion properties and spectral accelerations are shown in Table 4.3 and Figure 4.6, respectively.

Table 4.3. Ground Motion Set 2

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)
28	"Parkfield"	1966	6.19	SS	17.64	408.93	C	0.090
33	"Parkfield"	1966	6.19	SS	15.96	527.92	C	0.456
164	"Imperial Valley-06"	1979	6.53	SS	15.19	471.53	C	0.232

Table 4.3. (Cont.) Ground Motion Set 2

450	"Morgan Hill"	1984	6.19	SS	23.23	462.24	C	0.137
548	"Chalfant Valley-02"	1986	6.19	SS	21.55	370.94	C	0.276
551	"Chalfant Valley-02"	1986	6.19	SS	29.35	382.12	C	0.094
552	"Chalfant Valley-02"	1986	6.19	SS	22.08	456.83	C	0.188
553	"Chalfant Valley-02"	1986	6.19	SS	18.3	537.16	C	0.111
554	"Chalfant Valley-02"	1986	6.19	SS	18.3	537.16	C	0.111
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

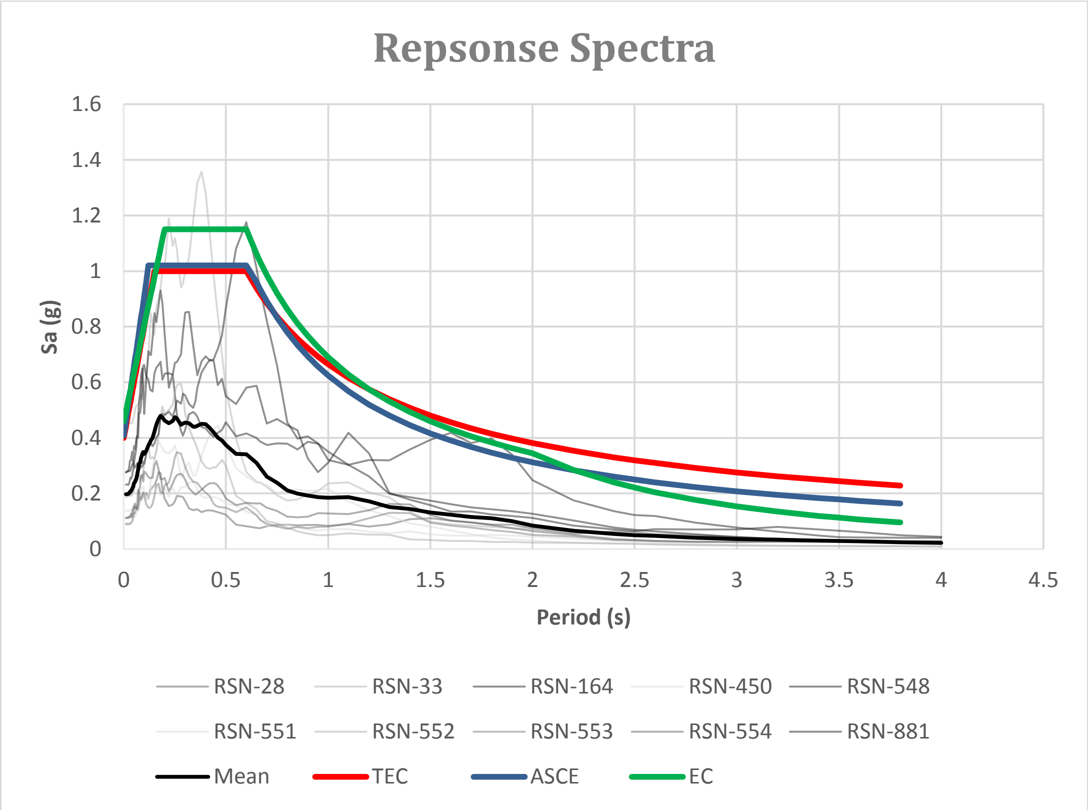

Figure 4.6. Response Spectra of Ground Motion Set 2

- Ground Motion Set 3

  This ground motion set contains only ground motions with normal faulting mechanism. Ten ground motions from four different earthquakes were employed. Nearly all spectral acceleration data of these ground motions are under 0.4g. The earthquake data and spectral accelerations are shown in Table 4.4 and Figure 4.7, respectively.

Table 4.4. Ground Motion Set 3

REC #	**EQ NAME**	**YEAR**	**Mw**	**FAULT TYPE**	**R_{jb} (km)**	**$V_{s,30}$ (m/sec)**	**SOIL TYPE**	**PGA (g)**
286	"Irpinia Italy-01"	1980	6.9	N	17.51	496.46	C	0.126
288	"Irpinia Italy-01"	1980	6.9	N	22.54	561.04	C	0.287
290	"Irpinia Italy-01"	1980	6.9	N	29.79	428.57	C	0.177
291	"Irpinia Italy-01"	1980	6.9	N	27.49	574.88	C	0.139
295	"Irpinia Italy-02"	1980	6.2	N	28.69	476.62	C	0.033
296	"Irpinia Italy-02"	1980	6.2	N	17.79	649.67	C	0.075
302	"Irpinia Italy-02"	1980	6.2	N	22.68	574.88	C	0.139
303	"Irpinia Italy-02"	1980	6.2	N	20.38	382	C	0.104
587	"New Zealand-02"	1987	6.6	N	16.09	551.3	C	0.375
815	"Griva Greece"	1990	6.1	N	26.75	454.56	C	0.067
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

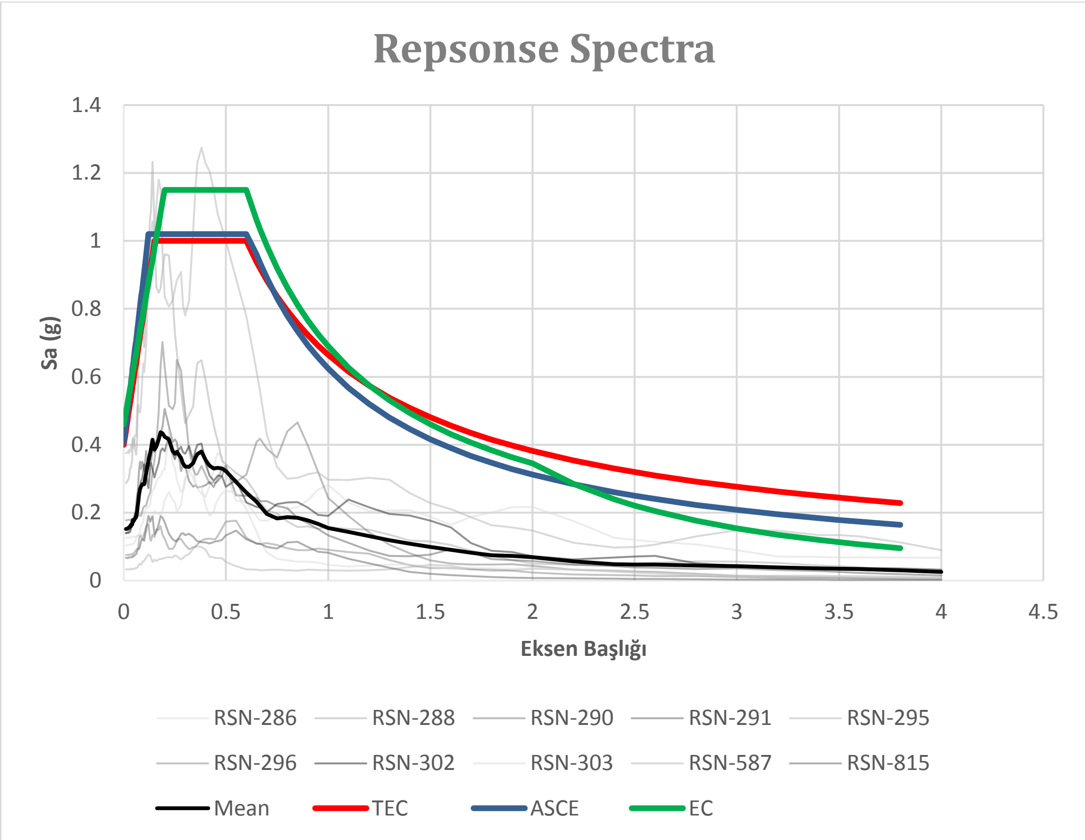

Figure 4.7. Response Spectra of Ground Motion Set 3

- Ground Motion Set 4

  This set contains ground motions with reverse fault mechanism. Three earthquakes generate these ten ground motions where the maximum peak ground acceleration of them is 0.43g. The ground motion properties and spectral accelerations are shown in Table 4.5 and Figure 4.8, respectively.

Table 4.5. Ground Motion Set 4

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430
63	"San Fernando"	1971	6.61	R	25.58	634.33	C	0.135
70	"San Fernando"	1971	6.61	R	22.23	425.34	C	0.190

Table 4.5. (Cont.) Ground Motion Set 4

72	"San Fernando"	1971	6.61	R	19.45	600.06	C	0.254
73	"San Fernando"	1971	6.61	R	17.22	670.84	C	0.226
8	"San Fernando"	1971	6.61	R	24.16	452.86	C	0.191
79	"San Fernando"	1971	6.61	R	25.47	415.13	C	0.149
88	"San Fernando"	1971	6.61	R	24.69	389	C	0.220
336	"Coalinga-01"	1983	6.36	R	27.1	541.73	C	0.118
340	"Coalinga-01"	1983	6.36	R	26.2	384.26	C	0.234
351	"Coalinga-01"	1983	6.36	R	28.72	450.61	C	0.120
359	"Coalinga-01"	1983	6.36	R	24.83	381.27	C	0.294
362	"Coalinga-01"	1983	6.36	R	29.01	438.74	C	0.120
369	"Coalinga-01"	1983	6.36	R	25.98	648.09	C	0.223
827	"Cape Mendocino"	1992	7.01	R	15.97	457.06	C	0.164
830	"Cape Mendocino"	1992	7.01	R	26.51	518.98	C	0.303
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

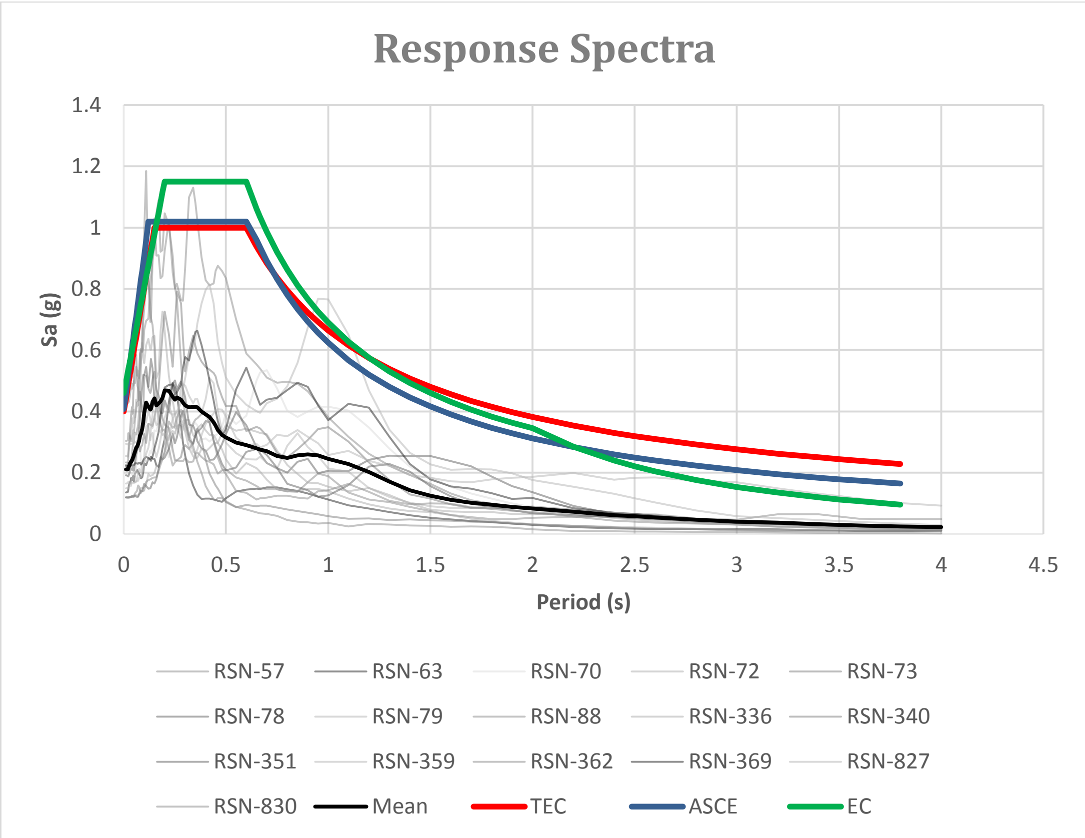

Figure 4.8. Response Spectra of Ground Motion Set 4

- Ground Motion Set 5

  This is the ground motion set which contains only ground motions with reverse oblique faulting mechanism. The set consists of ten ground motions from two earthquakes. The mean peak ground acceleration is approximately 0.25g. The earthquake information and spectral accelerations for the ground motions are shown in Table 4.6 and Figure 4.9, respectively.

Table 4.6. Ground Motion Set 5

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)
516	"N. Palm Springs"	1986	6.06	RO	27.21	425.17	C	0.227

Table 4.6. (Cont.) Ground Motion Set 5

521	"N. Palm Springs"	1986	6.06	RO	29.56	407.61	C	0.297
524	"N. Palm Springs"	1986	6.06	RO	23.2	379.32	C	0.084
534	"N. Palm Springs"	1986	6.06	RO	22.96	447.22	C	0.351
537	"N. Palm Springs"	1986	6.06	RO	16.55	659.09	C	0.187
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343
740	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.101
755	"Loma Prieta"	1989	6.93	RO	19.97	561.43	C	0.509
769	"Loma Prieta"	1989	6.93	RO	17.92	663.31	C	0.212
775	"Loma Prieta"	1989	6.93	RO	29.54	621.2	C	0.070
Mw: Moment Magnitude				Rjb: Joyner-Boore Distance				
Vs,30: Shear Wave Velocity				PGA: Peak Ground Acceleration				
SS: Strike Slip Mechanism				N: Normal Mechanism				
R: Reverse Mechanism				RO: Reverse Oblique Mechanism				

Examination of the five ground motion sets reveals that mean response spectra of ground motions is below the code response spectra. The closest mean spectra to the code spectra results from the set1. Therefore, it is expected that the most reasonable scaling would be obtained for the set1 and for this reason this set is used for the analyses.

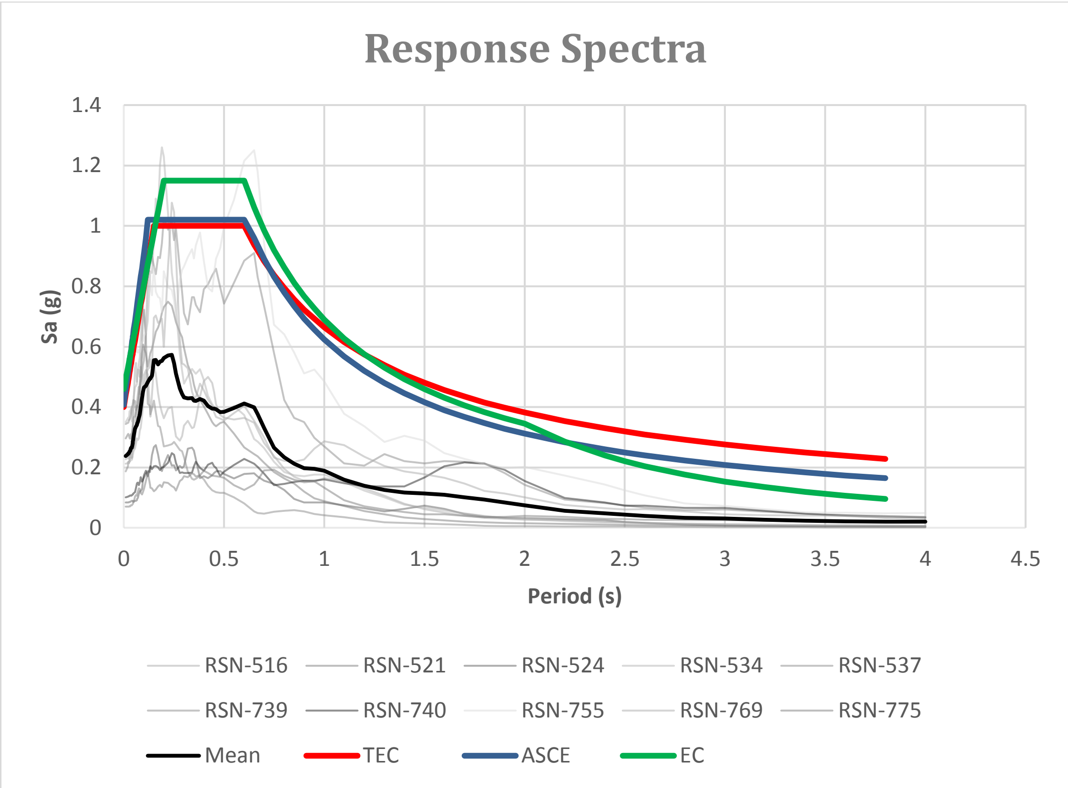

Figure 4.9. Response Spectra of Ground Motion Set 5

4.4. Scaling

As was defined in Chapter 2, scaling methods stated in TEC2007 and Eurocode-8 have almost the same context except the difference of duration of ground motion. Due to the reason that the selected ground motions for scaling have stationary part durations more than 15sec, only the scaling procedures identified in TEC2007 and ASCE 7-10 are utilized and compared in this study.

In order to capture the differences among the scaling procedures based on the different earthquake codes, the selected earthquake ground motions were scaled according to TEC2007 and ASCE 7-10. Each scaling procedure given in both codes, were applied to all three spectra for each frame. In other words, two scaling methods were applied to four frames according to the procedures defined in TEC and ASCE using all three response spectra defined in TEC2007, EC8 and ASCE7-10.

It should be highlighted that, there are no guidelines or requirements in the codes describing how the scaling should be applied. Thus, there is lack of clarification about whether each ground motion record should be scaled independently or a single scaling factor obtained based on the mean spectrum can be applied to all ground motions. Due to this reason, both approaches were needed to be considered in this study.

In scaling method 1, the following approach was used;

- The mean of spectral accelerations of scaled ground motions must be higher than 90% of the target response spectrum, for the given period range.
- To satisfy the first condition and not have divergent results with code based response spectra, spectral acceleration of each selected ground motion in the set were limited to be higher than 75% of the target elastic response spectrum for the given period range.
- The mean of spectral acceleration of selected ground motions at zero period must be larger than or equal to the peak ground acceleration of the target spectrum.

In this approach, separate scaling factors were obtained for each ground motion record. In scaling method 2, firstly the mean of ground motion set is obtained. Then, the scaling factor is determined through applying the criteria given in the codes to the mean response spectrum. Thus, a single scaling factor was determined and applied to all ground motions in the set.

In addition to the conditions given above, the number of records used in time history analysis is not fixed. In TEC2007, for example, at least 3 or 7 records can be used. Therefore, three subsets of ground motions from ground motion set 1 were defined to study the effect of the number of records. Table 4.7, Table 4.8 and Table 4.9 tabulate these subsets.

Table 4.7. Ground Motion Set 1.1

REC #	EQ NAME	YEAR	Mw	FAULTTYPE	R_{jb} (km)	$V_{s,30}$ (m/s)	SOIL TYPE	PGA (g)	Dur (s)
33	"Parkfield"	1966	6.19	SS	15.96	527.92	C	0.456	19.7
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430	16.8
164	"Imperial Valley-06"	1979	6.53	SS	15.19	471.53	C	0.232	36.4
288	"Irpinia Italy-01"	1980	6.9	N	22.54	561.04	C	0.287	19.4
359	"Coalinga-01"	1983	6.36	R	24.83	381.27	C	0.294	17.5
548	"Chalfant Valley-02"	1986	6.19	SS	21.55	370.94	C	0.276	16.6
587	"New Zealand-02"	1987	6.6	N	16.09	551.3	C	0.375	15.2
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343	25.3
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277	31.9
991	"Northridge-01"	1994	6.69	R	28.98	366.71	C	0.269	37.9

Table 4.8. Ground Motion Set 1.2

REC #	EQ NAME	YEAR	Mw	FAULT TYPE	R_{jb} (km)	$V_{s,30}$ (m/sec)	SOIL TYPE	PGA (g)	Dur (s)
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430	16.8
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343	25.3
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277	31.9

Table 4.9. Ground Motion Set 1.3

REC #	EQ NAME	YEAR	Mw	FAULTTYPE	R_{jb} (km)	$V_{s,30}$ (m/s)	SOIL TYPE	PGA (g)	Dur (s)
57	"San Fernando"	1971	6.61	R	19.33	450.28	C	0.430	16.8
164	"Imperial Valley-06"	1979	6.53	SS	15.19	471.53	C	0.232	36.4
359	"Coalinga-01"	1983	6.36	R	24.83	381.27	C	0.294	17.5
587	"New Zealand-02"	1987	6.6	N	16.09	551.3	C	0.375	15.2
739	"Loma Prieta"	1989	6.93	RO	19.9	488.77	C	0.343	25.3
881	"Landers"	1992	7.28	SS	17.36	396.41	C	0.277	31.9
991	"Northridge-01"	1994	6.69	R	28.98	366.71	C	0.269	37.9

Scaling factors determined for Ground Motion Set 1.1, Ground Motion Set 1.2 and Ground Motion Set 1.3 using both approaches are shown in Table 4.10, Table 4.11 and Table 4.12 respectively. The notation used in the table can be described as follows:

- M1 denotes Scaling Method 1 and M2 indicates Scaling Method 2.
- The acronym under FRAME column identifies the frames, code scaling procedure and the target response spectrum in the codes. For example, the first letter, given as a number, shows the number of story of the frame. The letter following that (T or A) denotes the scaling procedure in the code, T refers to TEC procedure and A refers to ASCE procedure. The last letter shows the target code spectrum, T indicates TEC, A stands for ASCE and E shows Eurocode.

Table 4.10. Scaling Factors According to Ground Motion Set 1.1

	SCALE FACTORS										
FRAME	**M1 R33**	**M1 R57**	**M1 R164**	**M1 R288**	**M1 R359**	**M1 R548**	**M1 R587**	**M1 R739**	**M1 R881**	**M1 R991**	**M2**
3TT	1.44	1.31	1.60	1.73	2.16	1.85	1.24	1.36	1.98	1.83	1.64
3TA	1.45	1.33	1.62	1.74	2.23	1.88	1.25	1.38	2.04	1.87	1.67
3TE	1.63	1.50	1.81	1.98	2.41	2.11	1.41	1.55	2.20	2.07	1.85
3AT	1.27	1.31	1.64	1.62	2.28	1.88	1.23	1.42	2.20	1.88	1.60
3AA	1.37	1.41	1.77	1.72	2.44	2.01	1.34	1.53	2.36	2.02	1.77
3AE	1.39	1.44	1.81	1.80	2.48	2.09	1.35	1.55	2.38	2.07	1.75
4TT	1.94	1.65	1.93	2.25	2.46	2.22	1.61	1.67	2.24	2.18	2.01
4TA	1.89	1.60	1.88	2.20	2.44	2.18	1.56	1.62	2.22	2.15	1.97
4TE	2.03	1.81	2.11	2.50	2.68	2.45	1.77	1.83	2.43	2.40	2.18
4AT	1.35	1.31	1.65	1.73	2.28	1.95	1.21	1.38	2.12	1.86	1.68
4AA	1.40	1.35	1.70	1.78	2.37	2.00	1.25	1.43	2.21	1.92	1.71
4AE	1.57	1.53	1.90	2.02	2.57	2.25	1.42	1.60	2.38	2.14	1.93
6TT	2.09	1.65	1.95	2.55	2.28	2.23	1.66	1.71	2.14	2.31	2.01
6TA	2.05	1.61	1.91	2.50	2.27	2.20	1.62	1.67	2.13	2.27	1.97
6TE	2.26	1.78	2.12	2.75	2.50	2.44	1.78	1.82	2.31	2.48	2.18
6AT	2.18	1.84	2.16	2.60	2.71	2.53	1.77	1.82	2.42	2.43	2.23
6AA	2.13	1.78	2.12	2.55	2.69	2.49	1.72	1.77	2.40	2.39	2.19

Table 4.10. (Cont.) Scaling Factors According to Ground Motion Set 1.1

6AE	2.34	1.96	2.32	2.83	2.91	2.74	1.89	1.94	2.58	2.62	2.42
8TT	2.62	1.90	2.16	3.19	2.19	2.46	1.84	1.91	2.01	2.67	2.28
8TA	2.24	1.69	1.95	2.76	2.10	2.26	1.64	1.71	1.91	2.36	1.97
8TE	2.51	1.91	2.20	3.10	2.35	2.54	1.86	1.93	2.14	2.66	2.18
8AT	2.47	1.78	2.09	3.17	2.28	2.49	1.84	1.90	2.15	2.66	2.23
8AA	2.36	1.80	2.11	2.99	2.39	2.51	1.82	1.89	2.20	2.59	2.19
8AE	2.63	2.02	2.36	3.34	2.65	2.81	2.04	2.11	2.45	2.89	2.42

Table 4.11. Scaling Factors According to Ground Motion Set 1.2

	SCALE FACTORS			
FRAME	**M1R57**	**M1R739**	**M1R881**	**M2**
3TT	1.31	1.36	1.98	1.49
3TA	1.49	1.54	2.20	1.66
3TE	1.36	1.41	2.06	1.53
3AT	1.41	1.52	2.30	1.65
3AA	1.57	1.69	2.52	1.85
3AE	1.42	1.54	2.37	1.69
4TT	1.36	1.38	1.95	1.56
4TA	1.51	1.53	2.13	1.65
4TE	1.44	1.46	2.06	1.69
4AT	1.42	1.49	2.23	1.63
4AA	1.62	1.69	2.47	1.83
4AE	1.47	1.55	2.33	1.67
6TT	1.77	1.83	2.26	1.99
6TA	1.67	1.73	2.19	1.84
6TE	1.85	1.89	2.38	2.03
6AT	1.57	1.55	2.15	1.81
6AA	1.60	1.59	2.22	1.76
6AE	1.71	1.69	2.33	1.94
8TT	1.92	1.93	2.03	1.99
8TA	1.75	1.77	1.97	1.84
8TE	1.94	1.96	2.17	2.03
8AT	2.10	2.22	2.47	2.22
8AA	1.87	1.96	2.27	2.04
8AE	2.12	2.21	2.55	2.26

Table 4.12. Scaling Factors According to Ground Motion Set 1.3

	SCALE FACTORS							
FRAME	**M1 R57**	**M1 R164**	**M1 R359**	**M1 R587**	**M1 R739**	**M1 R881**	**M1 R991**	**M2**
3TT	1.28	1.56	2.13	1.20	1.33	1.95	1.80	1.52
3TA	1.44	1.73	2.34	1.36	1.49	2.15	1.98	1.70
3TE	1.33	1.64	2.24	1.24	1.38	2.03	1.90	1.56
3AT	1.41	1.74	2.38	1.33	1.52	2.30	1.98	1.69
3AA	1.57	1.93	2.60	1.50	1.69	2.52	2.18	1.89
3AE	1.40	1.77	2.44	1.32	1.52	2.35	2.04	1.74
4TT	1.47	1.75	2.28	1.43	1.49	2.06	2.00	1.76
4TA	1.45	1.73	2.29	1.41	1.47	2.07	2.00	1.73
4TE	1.59	1.89	2.46	1.55	1.61	2.21	2.18	1.91
4AT	1.41	1.75	2.38	1.31	1.48	2.22	1.96	1.68
4AA	1.57	1.92	2.58	1.46	1.64	2.42	2.14	1.89
4AE	1.42	1.80	2.46	1.31	1.50	2.28	2.03	1.72
6TT	1.48	1.78	2.11	1.49	1.54	1.97	2.14	1.76
6TA	1.44	1.74	2.10	1.45	1.50	1.96	2.10	1.73
6TE	1.60	1.94	2.32	1.61	1.64	2.13	2.30	1.91
6AT	1.61	1.93	2.48	1.54	1.59	2.19	2.20	1.96
6AA	1.59	1.93	2.50	1.53	1.58	2.21	2.20	1.92
6AE	1.79	2.15	2.74	1.72	1.77	2.41	2.45	2.13
8TT	1.82	2.08	2.11	1.76	1.83	1.93	2.59	1.99
8TA	1.54	1.80	1.95	1.49	1.56	1.76	2.21	1.73
8TE	1.73	2.02	2.17	1.68	1.75	1.96	2.48	1.91
8AT	1.65	1.96	2.15	1.71	1.77	2.02	2.53	1.96
8AA	1.64	1.95	2.23	1.66	1.73	2.04	2.43	1.92
8AE	1.82	2.16	2.45	1.84	1.91	2.25	2.69	2.13

The scaling factors obtained for the ground motion set 1.1, set 1.2 and set 1.3 are also displayed in Figure 4.10, Figure 4.11 and Figure 4.12, respectively.

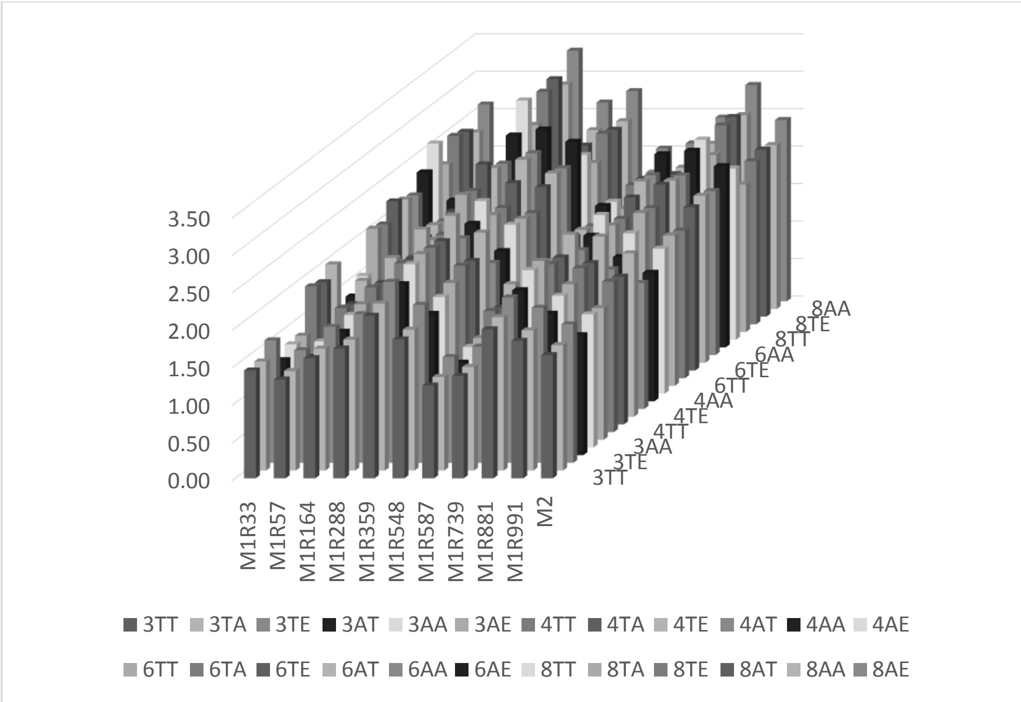

Figure 4.10. Scaling Factors Applied to Ground Motion Set 1.1.

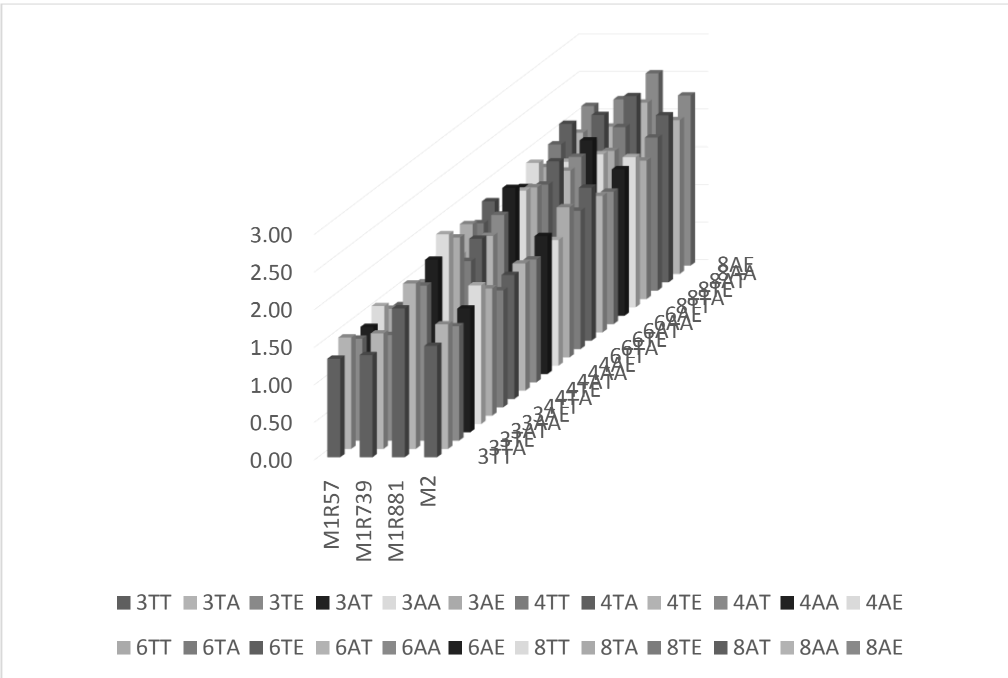

Figure 4.11. Scaling Factors Applied to Ground Motion Set 1.2.

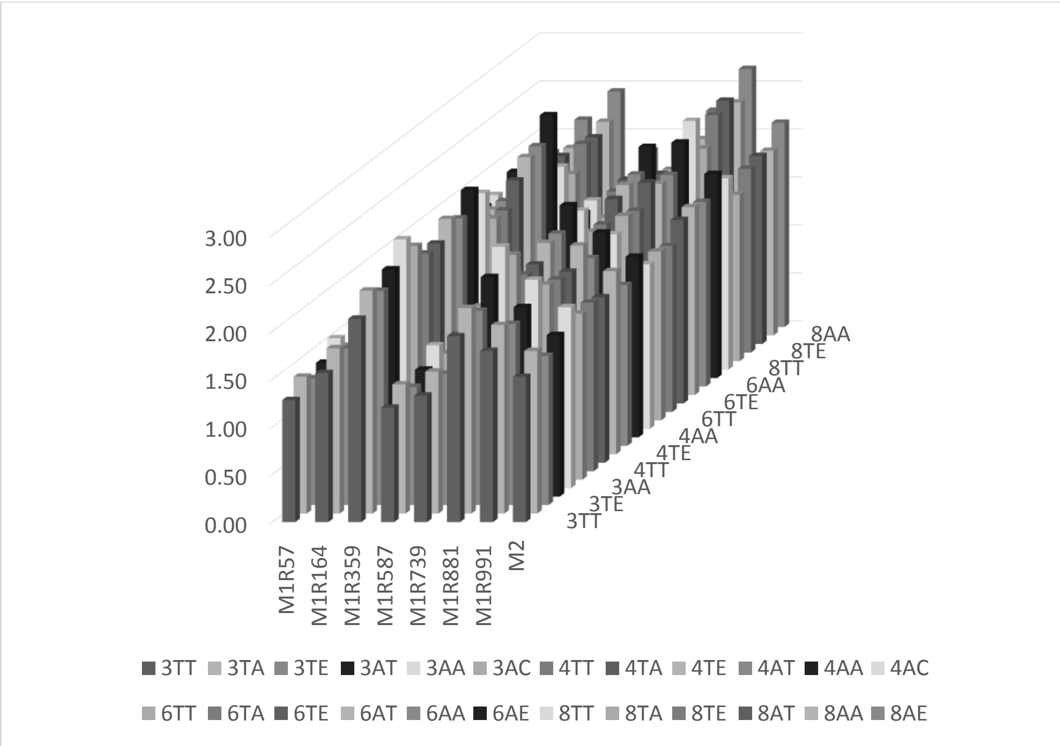

Figure 4.12. Scaling Factors Applied to Ground Motion Set 1.3.

Scaled spectral accelerations of selected ground motion sets 1.2 were calculated based on the procedures using two different scaling methodologies. These spectral accelerations and the code based response spectrum for each frame are depicted in Figures 4.13-4.16.

The acronym in the legends of figures stands for the mean of scaled spectral accelerations and the scaling method. For instance, GM1.1, GM1.2 or GM1.3 denotes the ground motion set 1.1, set 1.2 and set 1.3, respectively. The respective notation (M1 or M2) indicates the scaling method. In addition to these, first column of the figures demonstrates the scaled spectral accelerations according to TEC scaling procedure, while the second column shows spectral accelerations regarding the ASCE scaling procedure.

Figure 4.13 displays the mean of spectral accelerations of ground motion sets and the relation among code based response spectrum and the scaled accelerations for three story frame. Based on the results given in the figure the scaling method 2 gives slightly higher mean spectral acceleration values than scaling method 1 for ground motion set 1.1. On the other hand, for ground motion set 1.2 and set 1.3, scaling method 1 gives larger results than scaling method 2. Besides, when the spectral acceleration data of ground motion sets are compared, it can be noticed that ground motion set 1.2 has larger scaled spectral acceleration considering the TEC and ASCE response spectra. Contrarily, scaling of both ground motion sets gives closer spectral acceleration data when Eurocode based elastic response spectrum is used. For all six cases of time history analysis of three story frame, ground motion set 1.1 and set 1.3 have closer spectral acceleration data.

When the mean of scaled spectral accelerations for four story frame are compared to each other, scaling method 2 gives slightly higher results for ground motion set 1.2, similar to three story frame results. Although there is no clear indication on which ground motion set gives higher spectral accelerations for four story frame, it can be stated that the ground motion set 1.3 has smaller spectral acceleration data based on the TEC scaling procedure and the ground motion set 1.2 creates slightly higher results based on the ASCE scaling procedure. The spectral acceleration data obtained for four story frame are demonstrated in Figure 4.14.

The mean spectral accelerations of six story frame are presented in Figure 4.15. As it can be clearly seen in the figure, scaling method 1 and 2 create almost the same spectral acceleration data from all ground motion sets respectively. When spectral acceleration values obtained from ground motion set 1.1 and set 1.2 are compared to each other, the difference among ground motion set 1.1 and set 1.2 gets closer in comparison to three and four story frames. Moreover, gound motion set 1.3 creates the lowest spectral accelerations based on both TEC and ASCE scaling procedure.

Figure 4.16 shows the relation among the mean of scaled spectral acceleration of ground motion sets and code-defined response spectrum for eight story frame. Similar to all other frames, scaling method 1 and 2 give nearly the same mean of scaled response spectra for both ground motion set 1.1 and set 1.2, whreas the ground motion set 1.3 has the smallest spectral acceleration data for eight story frame. Satisfying the condition at the inclined part of the code-defined response spectrum predominates the scaling procedure.

As a result of scaling part, it can be concluded that scaling method 1 and 2 do not generate a significant difference in mean of spectral acceleration of scaled ground motion records. Moreover, the effect of using more, equal or less than 7 ground motion records could not be seen clearly from the scaled spectral acceleration data. Therefore, as the next step of the study, the scaled ground motions were applied to the frames and the analysis results were explored.

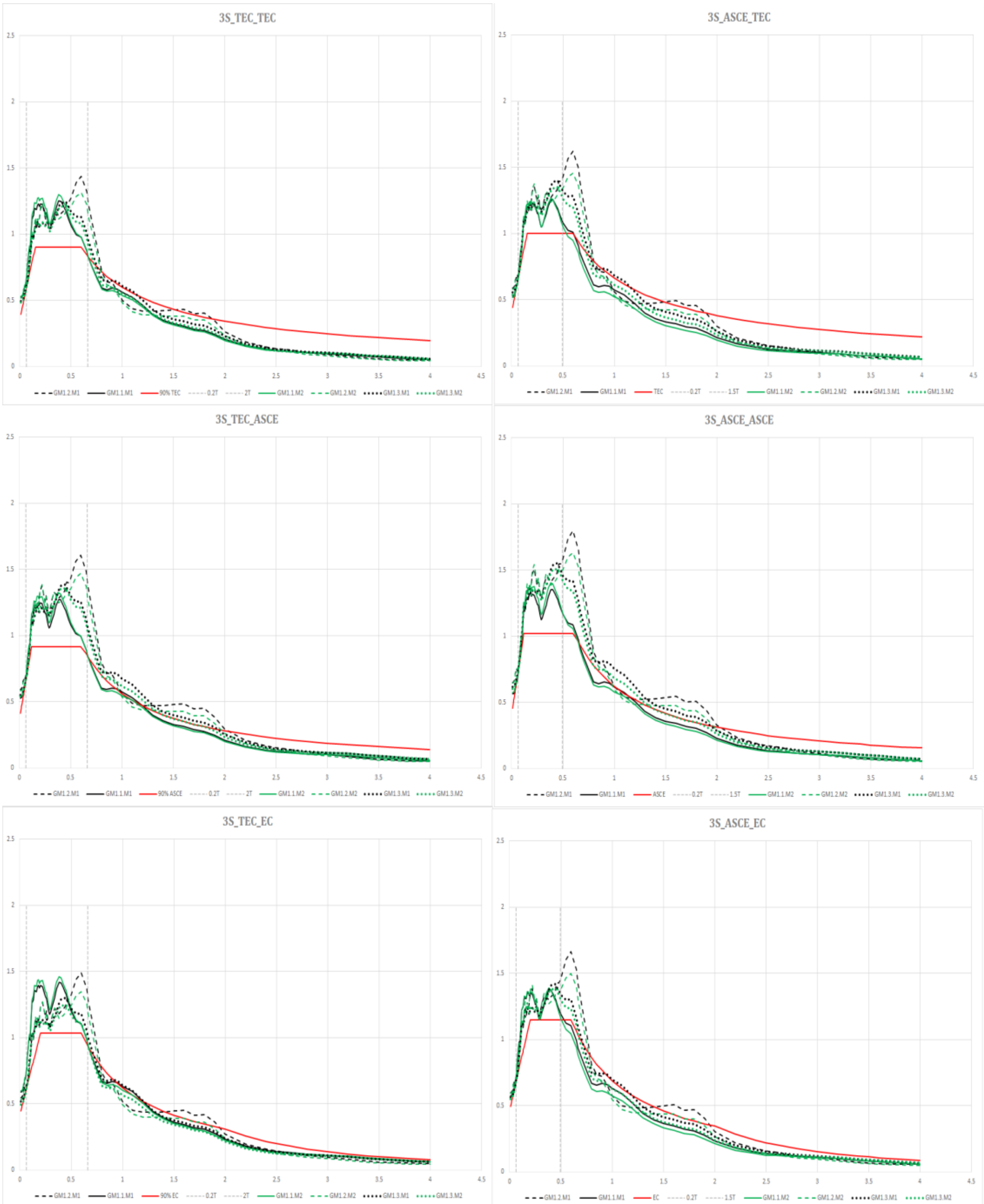

Figure 4.13. Means of Scaled Spectral Accelerations for 3-story frame

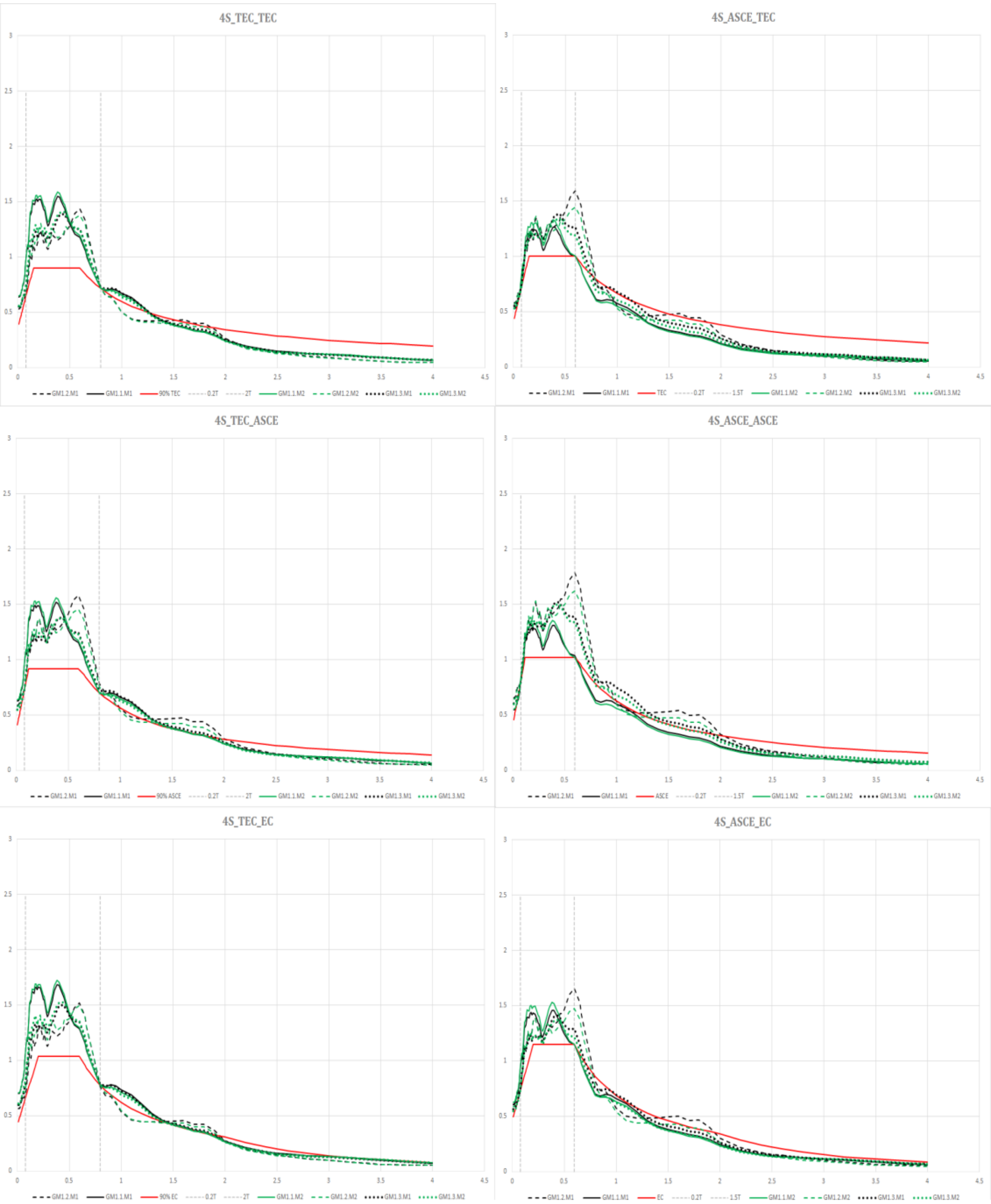

Figure 4.14. Means of Scaled Spectral Accelerations for 4-story frame

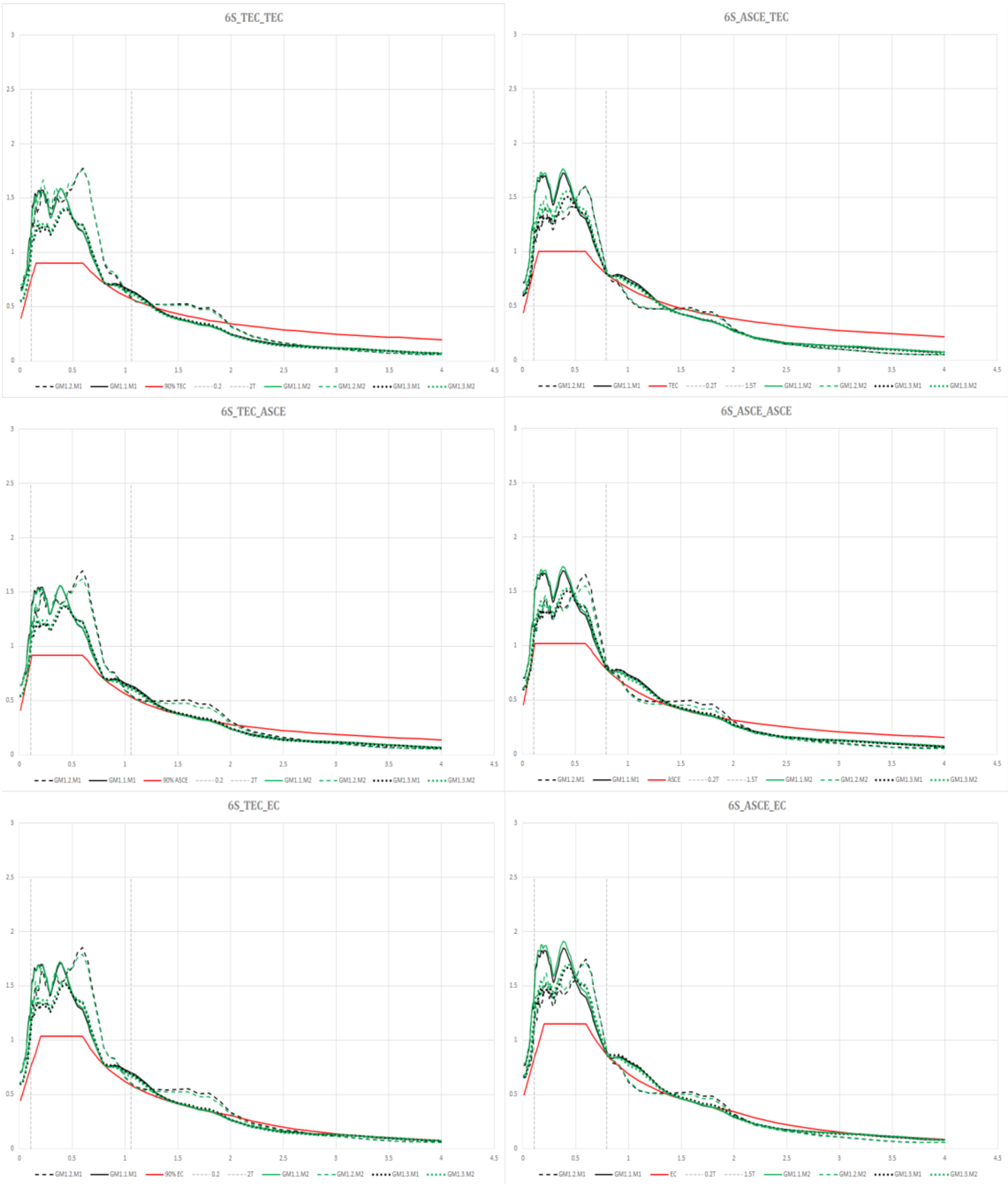

Figure 4.15. Means of Scaled Spectral Accelerations for 6-story frame

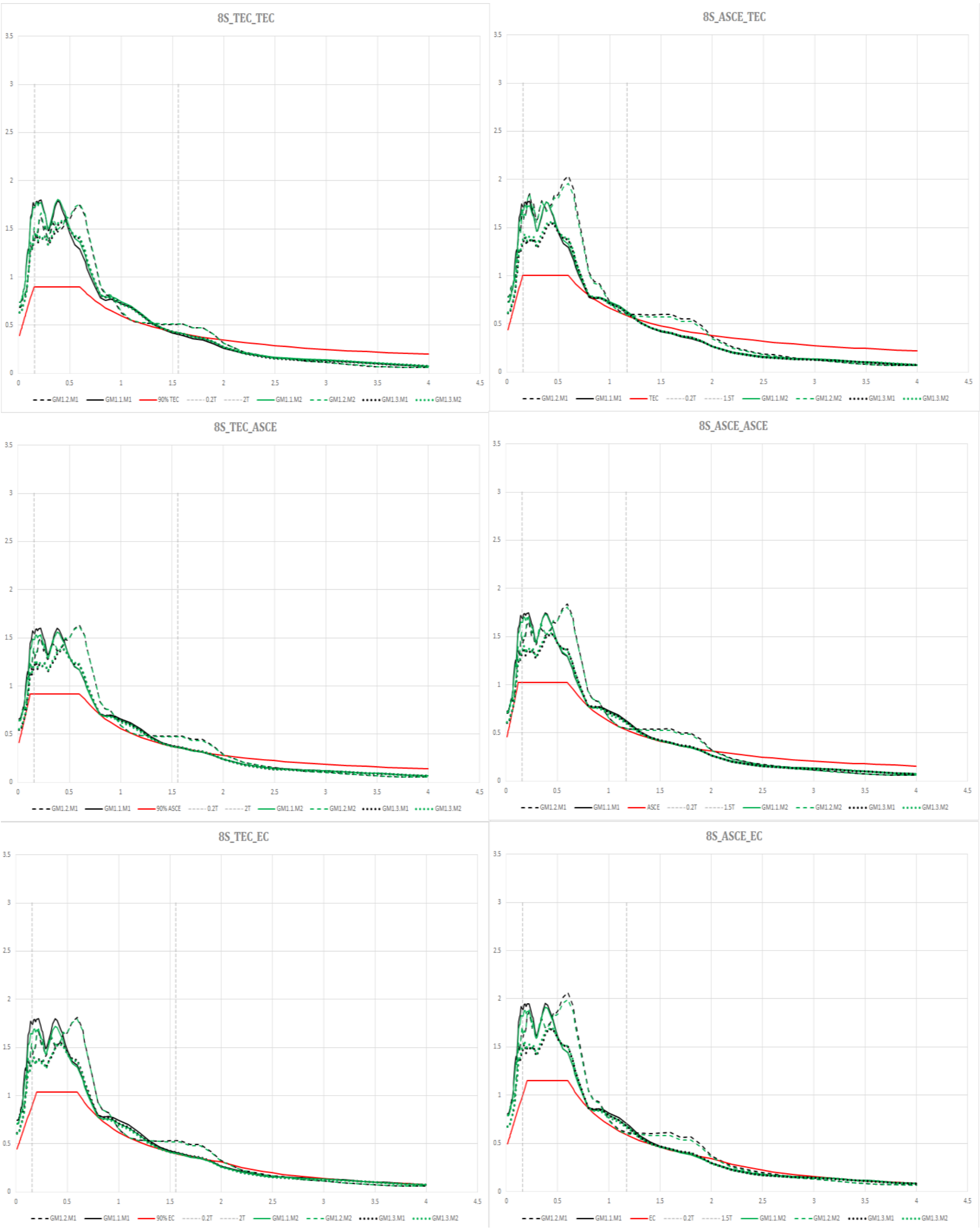

Figure 4.16. Means of Scaled Spectral Accelerations for 8-story frame

4.4. Analyses of Buildings

After scaling of selected ground motion sets were completed, linear time history analyses of all four frames were performed. The analyses were carried out by using all ground motion sets separately. Only scaling method 1 was utilized to scale ground motions due to not having a significant difference between the two scaling methods. Each scaled ground motion is applied to each frame. Because there are more than seven ground motion records in set 1.1 and seven ground motion records in set 1.3, the mean of the analyses results is considered for these sets. On the other hand, the maximum result is taken into account when ground motion set 1.2 is used in the analyses.

The next step after linear time history analyses is to perform response spectrum analyses of the frames for comparison purposes. All three response spectra given in TEC, ASCE and EC separately were used in these analyses.

The maximum roof displacements obtained from the analyses were utilized to compare scaling procedures and to see the effects of ground motion sets on linear time history analysis.

The maximum roof displacements of each frame obtained from linear time history analyses based on each ground motion sets and from response spectrum analyses are shown in Table 4.13. The notation under FRAME column in Table 4.13 identifies the frames and the target response spectrum in the codes. The first letter, given as a number, shows the number of story of the frame. The letter following that denotes the target code spectrum, T indicates TEC, A stands for ASCE and E shows Eurocode.

Table 4.13. Maximum Roof Displacements of All Frames

FRAME	Maximum Roof Displacements (m)						
	TEC Procedure			ASCE Procedure			CODE
	GM1.1	GM1.2	GM1.3	GM1.1	GM1.2	GM1.3	
3T	0.040	0.052	0.039	0.040	0.056	0.044	0.036
3A	0.041	0.059	0.044	0.043	0.062	0.049	0.036
3E	0.046	0.054	0.041	0.044	0.057	0.044	0.041
4T	0.082	0.070	0.074	0.067	0.080	0.074	0.054
4A	0.080	0.077	0.074	0.070	0.089	0.081	0.055
4E	0.089	0.074	0.080	0.077	0.084	0.076	0.062
6T	0.116	0.211	0.118	0.127	0.201	0.128	0.093
6A	0.114	0.205	0.116	0.125	0.208	0.128	0.095
6E	0.126	0.223	0.128	0.136	0.218	0.142	0.107
8T	0.168	0.212	0.174	0.168	0.257	0.170	0.167
8A	0.152	0.205	0.152	0.168	0.237	0.169	0.164
8E	0.171	0.226	0.170	0.188	0.266	0.187	0.182

When the roof displacement of 3-story frame are examined, it is clearly seen that using less than seven ground motion in the analyses results in the most unfavorable roof displacements for all cases. Moreover, using seven or more than seven ground motion records does not affect the analyses results significantly for 3-story frame. In addition, response history analyses give smaller results than linear time history analyses. The linear time history analyses using ground motion set 1.1 for TEC scaling procedure give similar results for 3-story frame. On the other hand, while the results for ground motion set 1.2 are compared with each other, minimum roof displacements occur when TEC spectrum was used as the basis of the analyses and maximum one is observed when ASCE response spectrum is used. Besides, ground motion set 1.3 creates smaller roof displacements when TEC scaling procedure is used. When fundamental period of the structure is around 0.3~0.35 seconds, it can be said that not the scaling procedure nor the the reference response spectrum but the number of ground motion records used in the analyses affects the results significantly.

The displacement results of 3-story frame are presented in Figure 4.17.

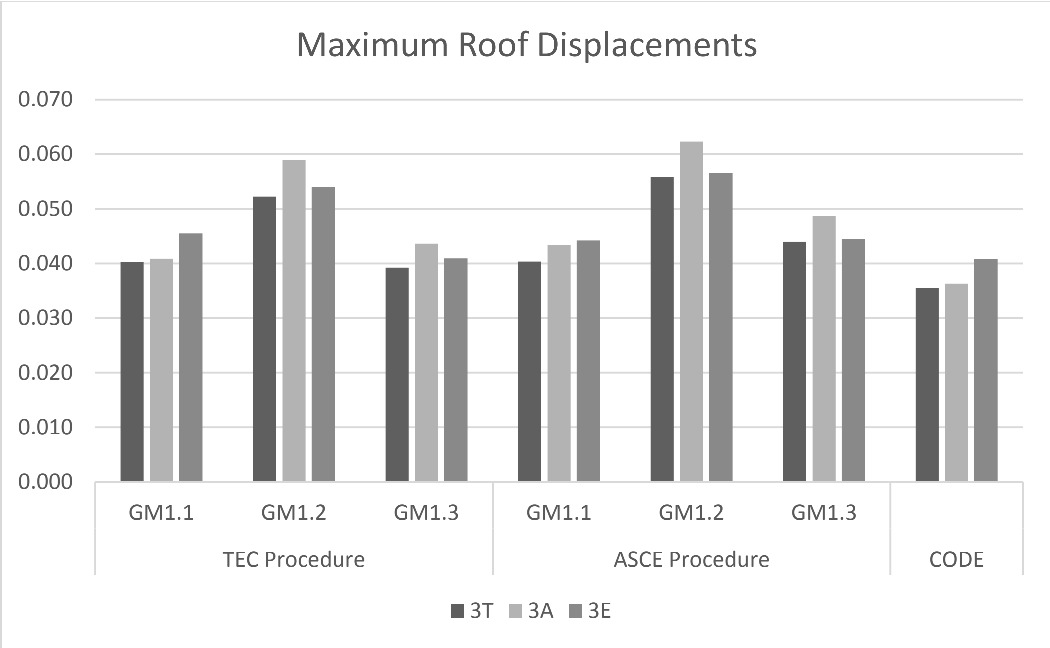

Figure 4.17. Maximum Roof Displacements of 3-Story Frame

Maximum roof displacements of 4-story frame are shown in Figure 4.18. As it is seen from the figure, response spectrum analyses give the smallest results. It can not be claimed that the number of ground motion records in the analyses affects the results positively or negatively. For example, ground motion set 1.1 creates higher roof displacements utilizing ground motions scaled with respect to TEC procedure. On the contrary, the same ground motion set causes lower roof displacements in the analyses when ground motions are scaled according to ASCE procedure. The distribution of maximum roof displacements obtained from response spectrum analyses shows similarity with the sequence of spectra, which are the basis of analyses. On the other hand, no connection was observed between the displacement results of linear time history and their base spectrum, for 4-story frame. Moreover, the difference between results of linear time history analyses using all three ground motion is relatively smaller compared to the difference in results of other frames. Therefore, it can be claimed that the supreme factor which affects the analyses results is the type of analyses rather than the number of ground motions, type of scaling and the reference response spectrum for structures with fundamental period is around 0.4 seconds.

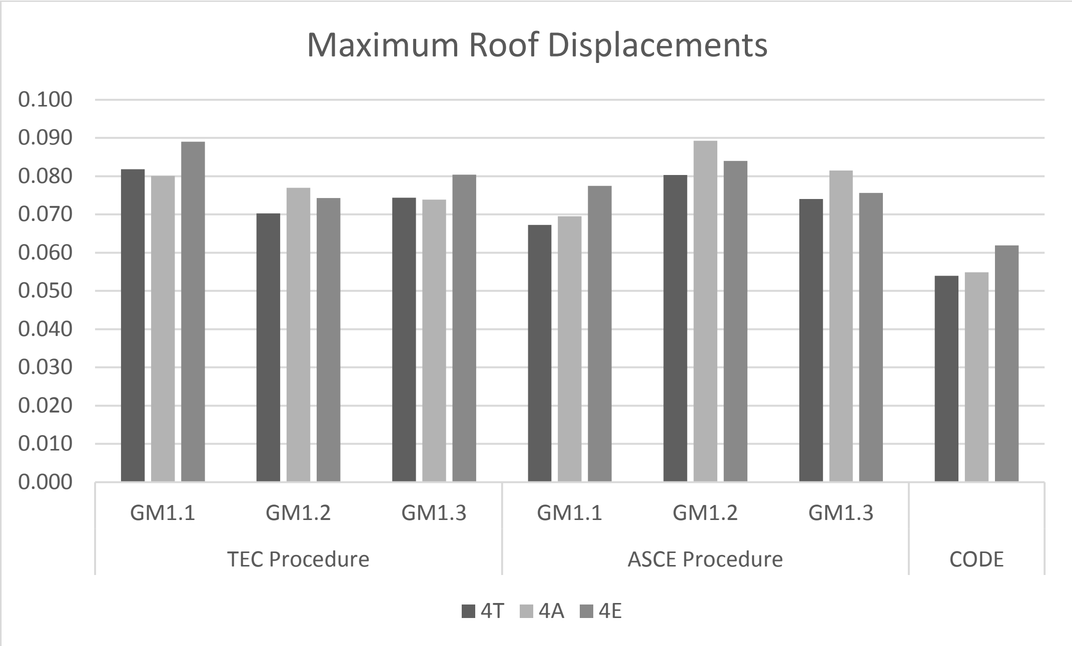

Figure 4.18. Maximum Roof Displacements of 4-Story Frame

There is a significant difference in analyses results considering the number of records used for 6-story frame. Ground motion set 1.2 creates nearly two times higher roof displacements than ground motion set 1.1 and set 1.3 in time history analyses. This situation most probably depends on the restriction defined in codes which permits to use the mean of results if seven or more ground motion records are used in the analyses. The ground motion set 1.1 and set 1.3 give nearly the same results for 6-story frame. Moreover, the analyses based on TEC elastic response spectrum and ASCE response spectrum nearly give the same results while the analyses based on EC spectrum create higher results. Similar to 3-story and 4-story frame displacements, response spectrum analyses give the smallest displacements. In addition to this, TEC scaling procedure and ASCE scaling procedure do not affect the results of linear time history analyses considerably.

The roof displacements of 6-story frame are presented in Figure 4.19.

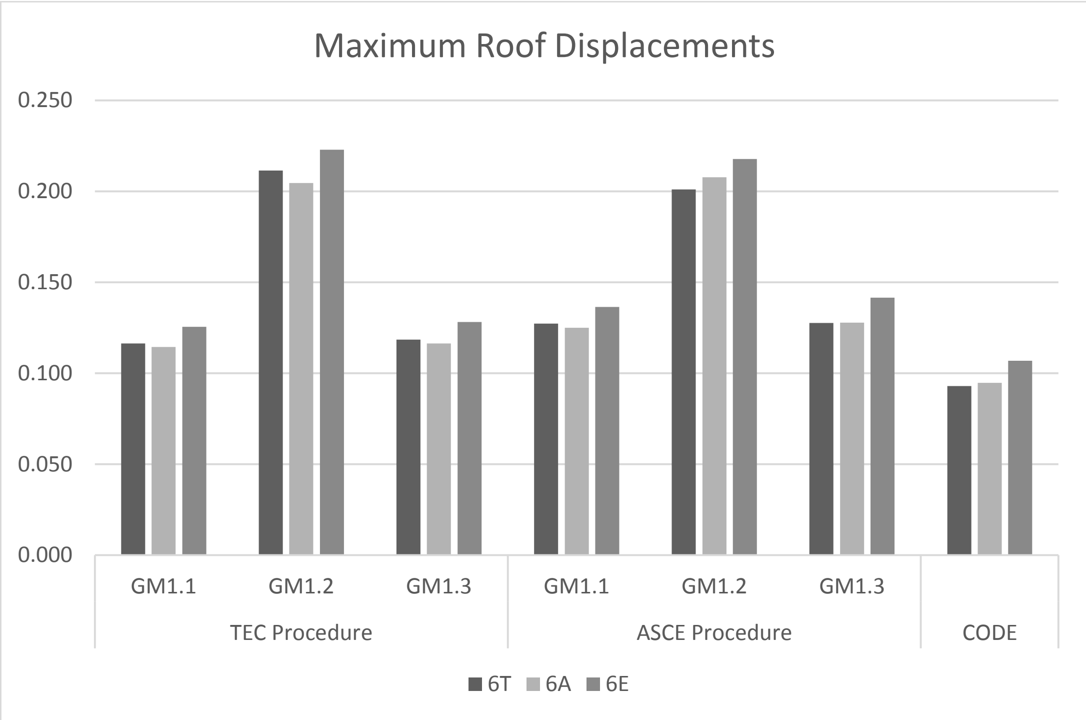

Figure 4.19. Maximum Roof Displacements of 6-Story Frame

When the maximum roof displacements of 8-story frame are reviewed, unlike the results of other frames, it is seen that the linear time history analyses using ground motion set 1.1 and set 1.3 were used and response spectrum analyses give similar results. The analyses that use ground motion set 1.2 give the highest roof displacements similar to the other frames. In other words, using more than seven ground motions or less than seven ground motions creates considerably major change in analyses results when the fundamental period of the building is around 0.8 seconds. Analyses based on ASCE response spectrum gives relatively smaller results than analyses based on TEC and EC response spectra. Besides, the difference in scaling procedure does not affect the results significantly.

The roof displacement results of 8-story frame are shown in Figure 4.20.

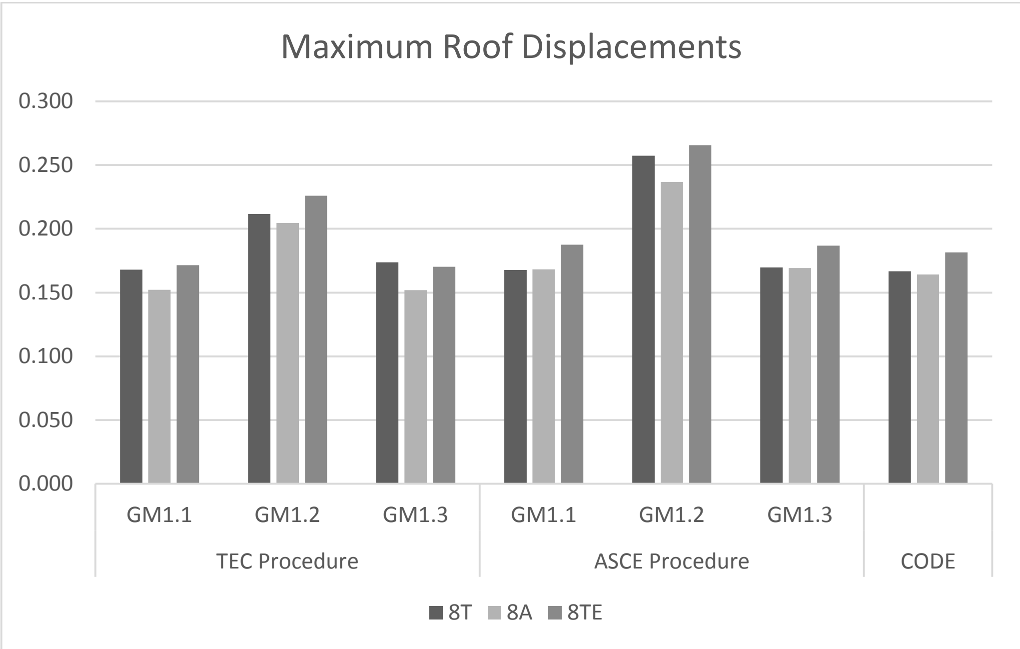

Figure 4.20. Maximum Roof Displacements of 8-Story Frame

The result of frame analyses can be summarized as follows:

- It can be concluded that the TEC scaling procedure and the ASCE scaling procedure give similar results for 3, 6 and 8 story frames, notwithstanding to the ground motion record set used in the analyses. For 4-story frame, the change in scaling procedure affects the results.
- Ground motion set 1.1 and set 1.3 givi similar results and lead to smaller values than ground motion set 1.2 for 3, 6 and 8 story frames. The opposite of this

situation appears in linear time history analyses of 4-story frame using scaling based on ASCE procedure and with ground motion set 1.1.

- For all frames, response spectrum analyses give the smallest roof displacement results.
- The difference between time history analyses and response spectrum analyses become closer as the fundamental period of the building increases.
- It can be concluded that the number of ground motions affects the analyses results much more than the type of the scaling procedure, the method of scaling and the base spectrum used for scaling.
- Using seven or more ground motion record does not affect analyses results considerably. Therefore, it can be said that for reducing the effort and time used in time history analyses, selecting, scaling and using seven ground motion can be preferred.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1. Summary

This study has been conducted to provide the effects of ground motion selection and scaling on time history analyses and the differences of scaling procedures in earthquake codes and their effects on structural response of low-rise and mid–rise reinforced concrete frame structures in Turkey.

The design of 3, 4, 6 and 8–story moment resisting reinforced concrete frames were carried out according to the current codes. The analytical models were formed with respect to the specific characteristics of construction practice and the observed seismic performance after major earthquakes in Turkey. Since the main objective of this study is to observe the effect of scaling procedures on seismic design of buildings, similar material properties and loading were applied on the buildings. Two dimensional finite element models of the building were used in the analyses.

The ground motion record set for scaling is taken from PEER earthquake motion database. 51 ground motion records are selected as candidate. 5 ground motion sets are formed and the most suitable to scaling is selected. To see the differences of scaling procedures more clearly, three subsets are created, first of them has ten ground motion records, second one has three ground motion records and the other one has seven ground motion records. Two scaling methods were used for each scaling procedure to define whether scaling methods affect the results significantly or not. These ground motions are scaled for each frame using each code scaling procedure.

Final part of the study is devoted to the time history analyses of the reinforced concrete frame structures considered. Moreover, the code defined response spectrum analyses were performed for each building.

5.2. Conclusions

Based on the results obtained the following conclusions can be drawn:

• The time history analyses results strongly depend on the ground motion recording selection and the scaling procedure. These choices can cause significant discrepancies in the response of the structures. The number of ground motion records can be identified as the most critical parameter that affects the results.

• To have seven or more ground motion records in analyses gives less conservative results with respect to having less than seven ground motion records for all cases and all frames except the 4-story frame.

• The TEC scaling procedure and the ASCE scaling procedure create similar results for 3, 6 and 8 story frames, whereas the change in scaling procedure shows variation in analyses results for 4 story frame.

• It was seen that using different scaling methods for applying code based scaling procedures does not affect the scaling result significantly.

• The scaled ground motions result in higher response spectra than the code-defined response spectra. Therefore, the results of seismic design by using scaled ground motions give more conservative results for all fundamental periods.

• It is accepted that time history analyses give more accurate results than response spectrum analyses. On the other hand, scaling of ground motion records leads to more conservative results. This is due to having the mean spectrum that is larger than the reference code spectrum.

5.3. Recommendation for Future Studies

The differences and effects of scaling procedures with respect to different earthquake design codes are reflected in this study. Therefore, the scaling is applied to ground motion data set and these data sets are used in seismic design of low and mid-rise buildings. Nevertheless, some further investigations could be conducted. Since this study is limited to 3, 4, 6, and 8–story reinforced concrete moment resisting frame buildings, future scaling studies can be conducted for high rise buildings, shear wall buildings, steel buildings and also masonry buildings. Infill walls can be also included into the analytical models. Besides, only one ground motion data set is used to see the difference between scaling procedures in several earthquake design codes in this research. More than one earthquake motion sets can be selected and more than three subsets are created, then scaling is applied to these sets for one procedure to observe only the ground motion selection. Some irregularities in plan and elevation can be considered.

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APPENDIX A

REINFORCEMENT DETAILS OF FRAMES

Explanation and details of reinforcement of columns and beams of each frame are given in Chapter 3. Each of the reinforcement details given in Figure A.1 to Figure A.70 are used for modeling of the frames. The following given figures are directly taken from PROBINA software output pages.

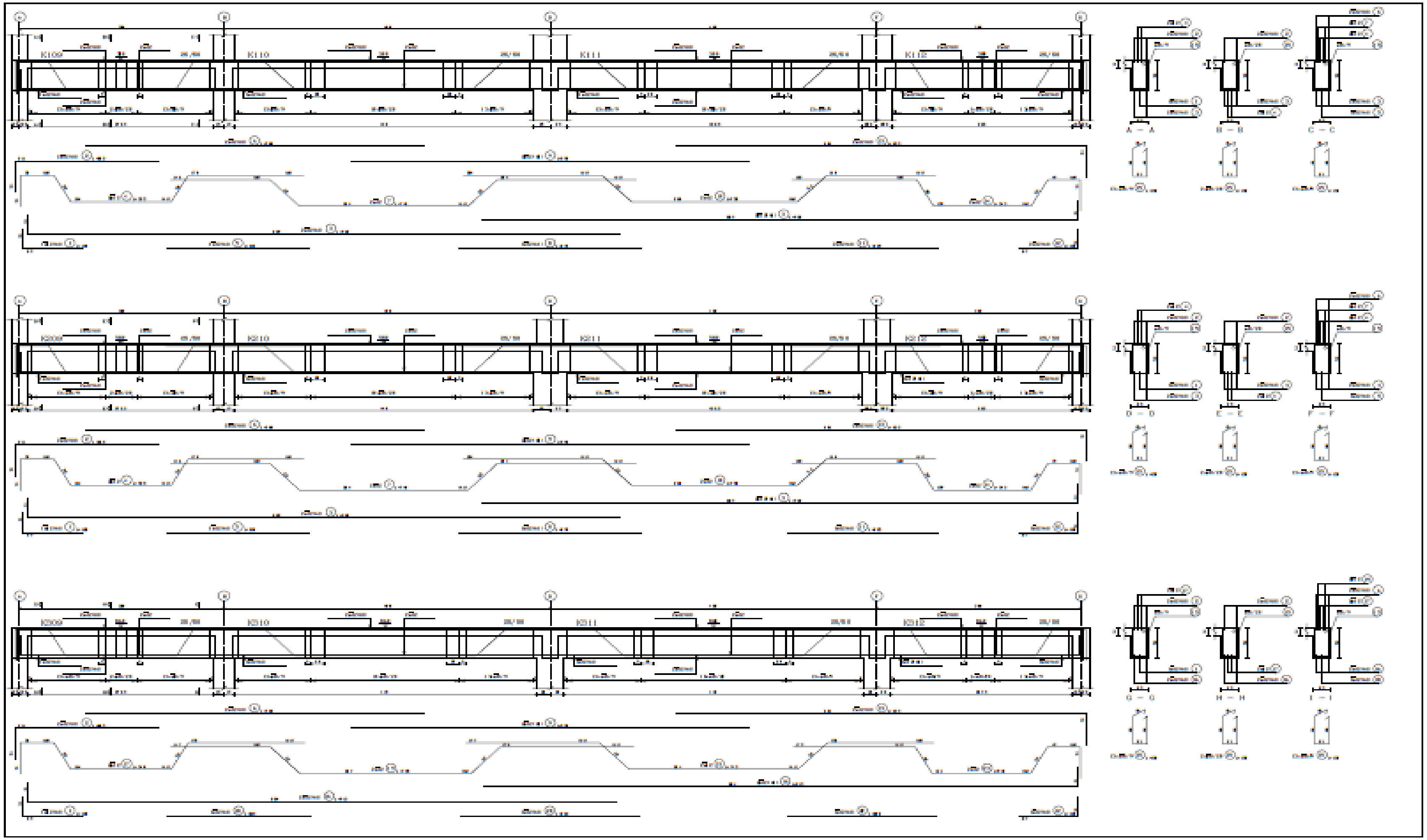


Figure A.1: Beam Details of 3-Story Frame

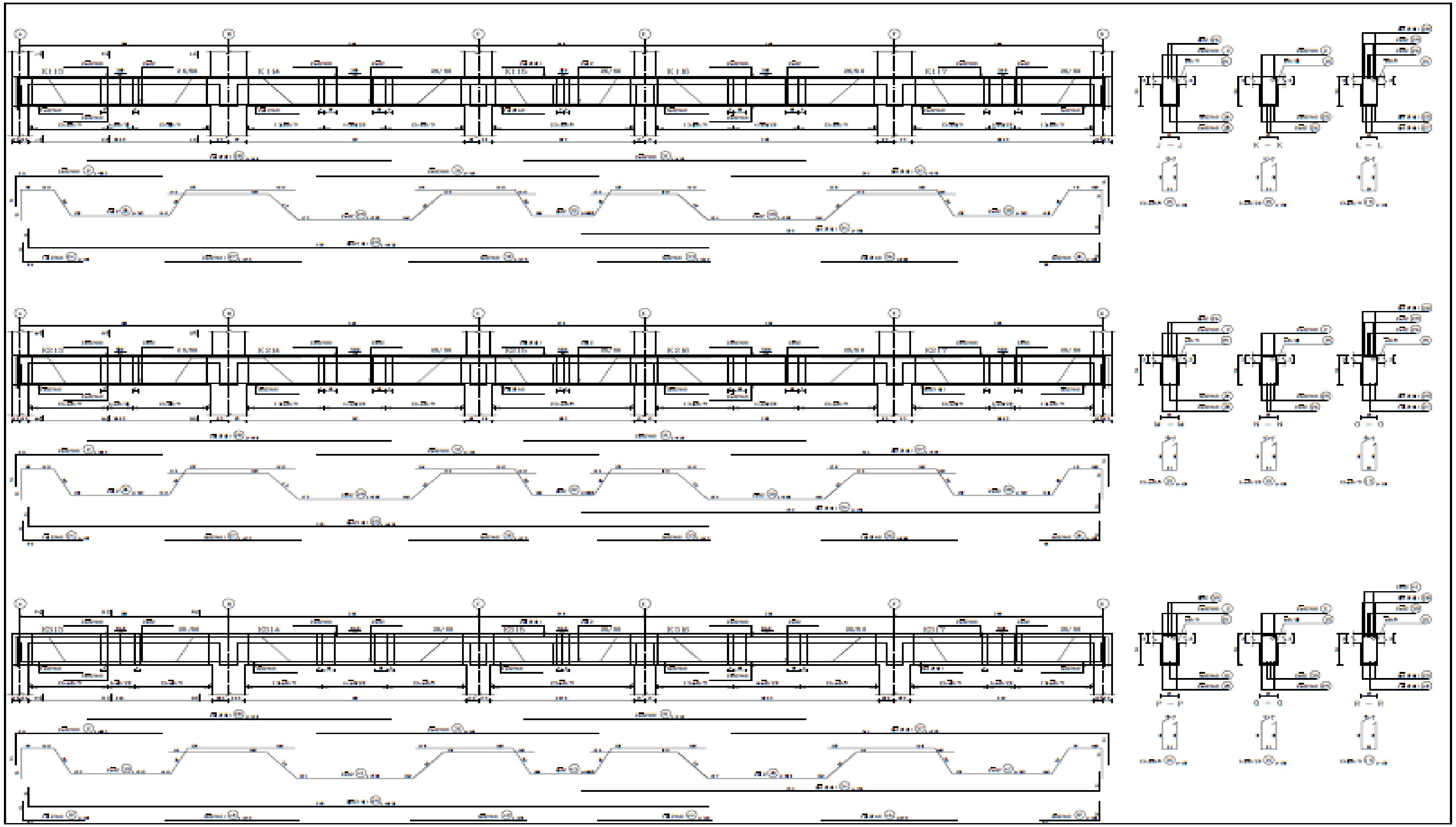


Figure A.2: Beam Details of 3-Story Frame

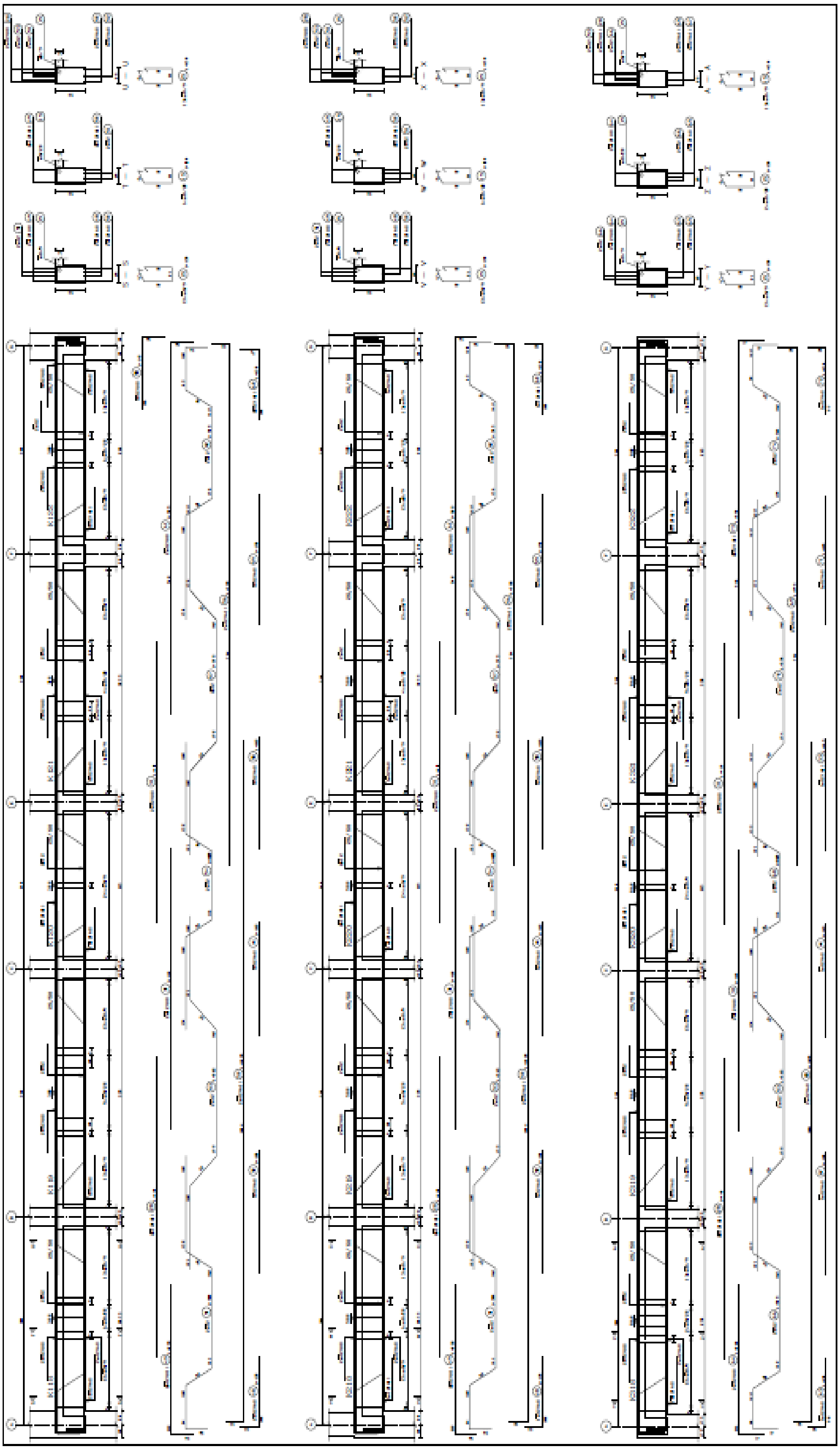


Figure A.3: Beam Details of 3-Story Frame

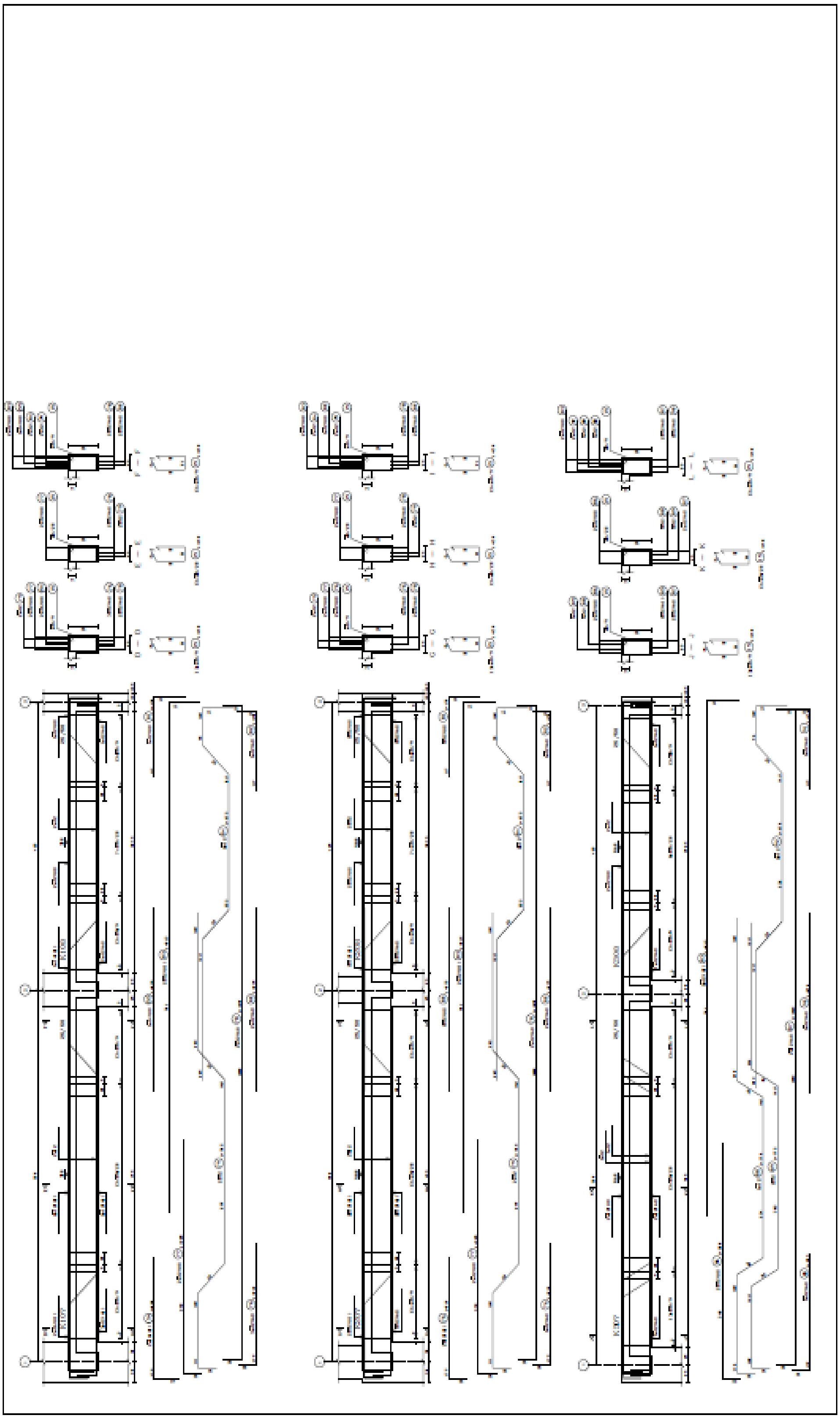


Figure A.4: Beam Details-of 3-Story Frame

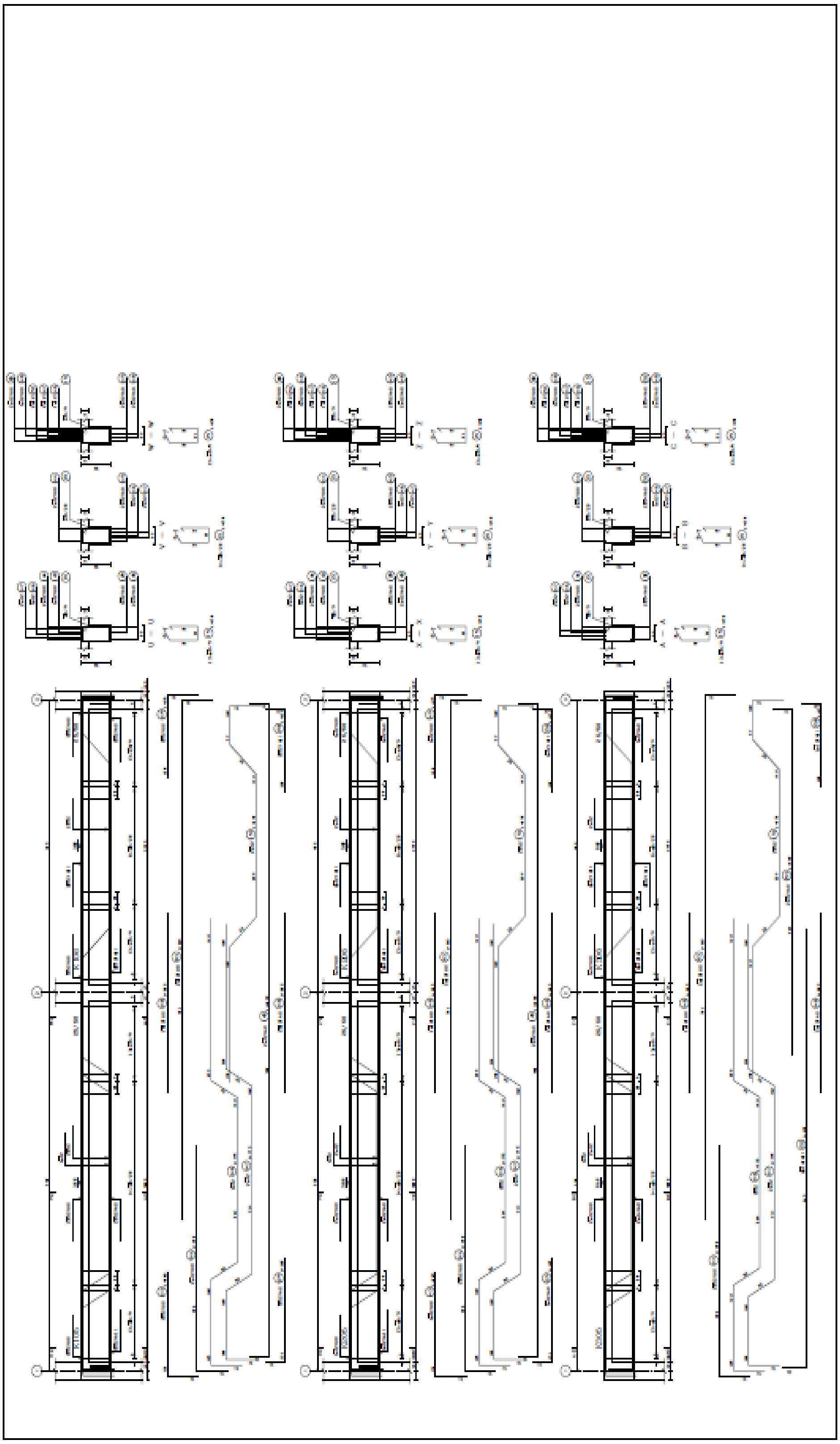


Figure A.5: Beam Details of 3-Story Frame

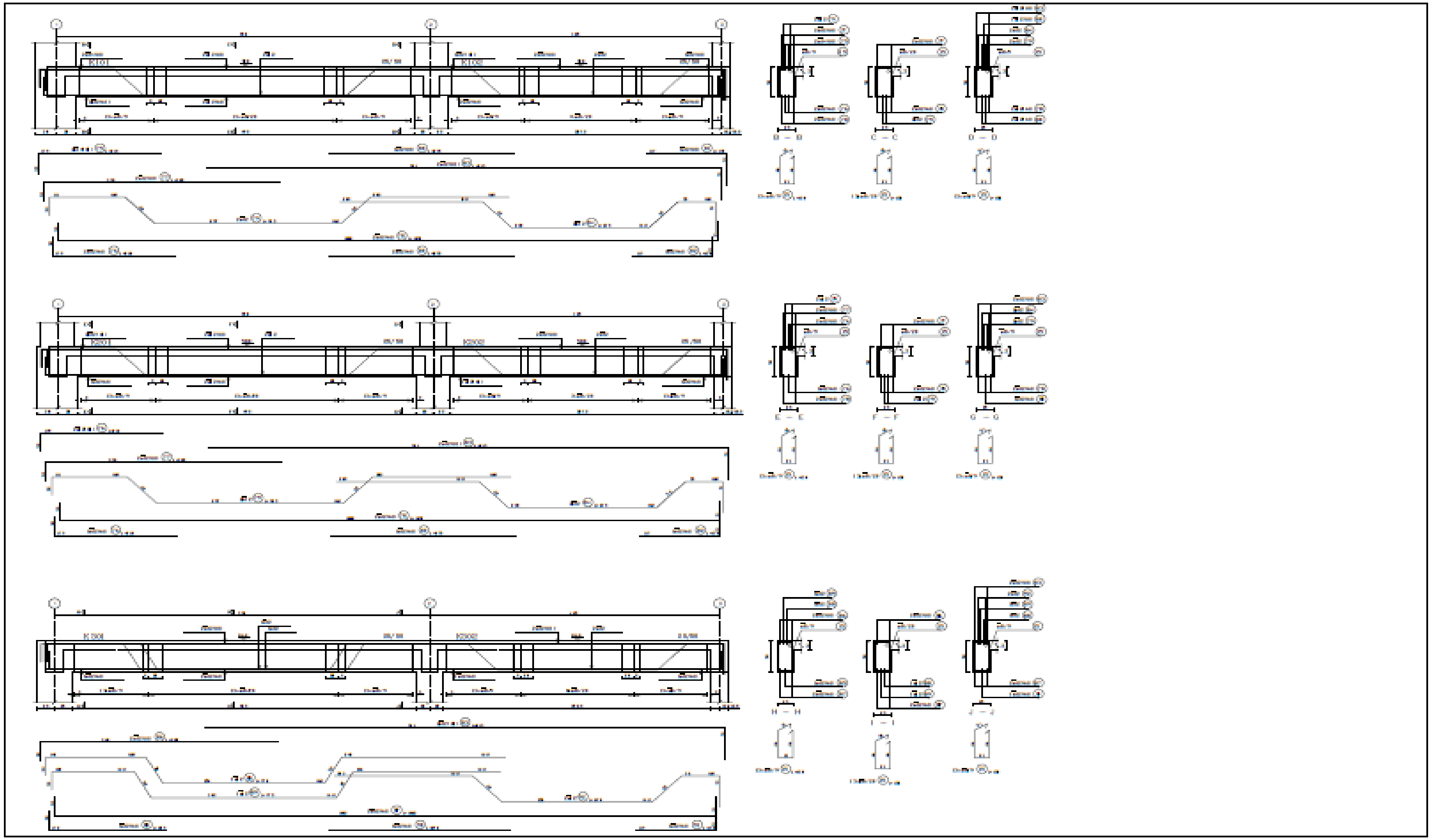


Figure A.6: Beam Details of 3-Story Frame

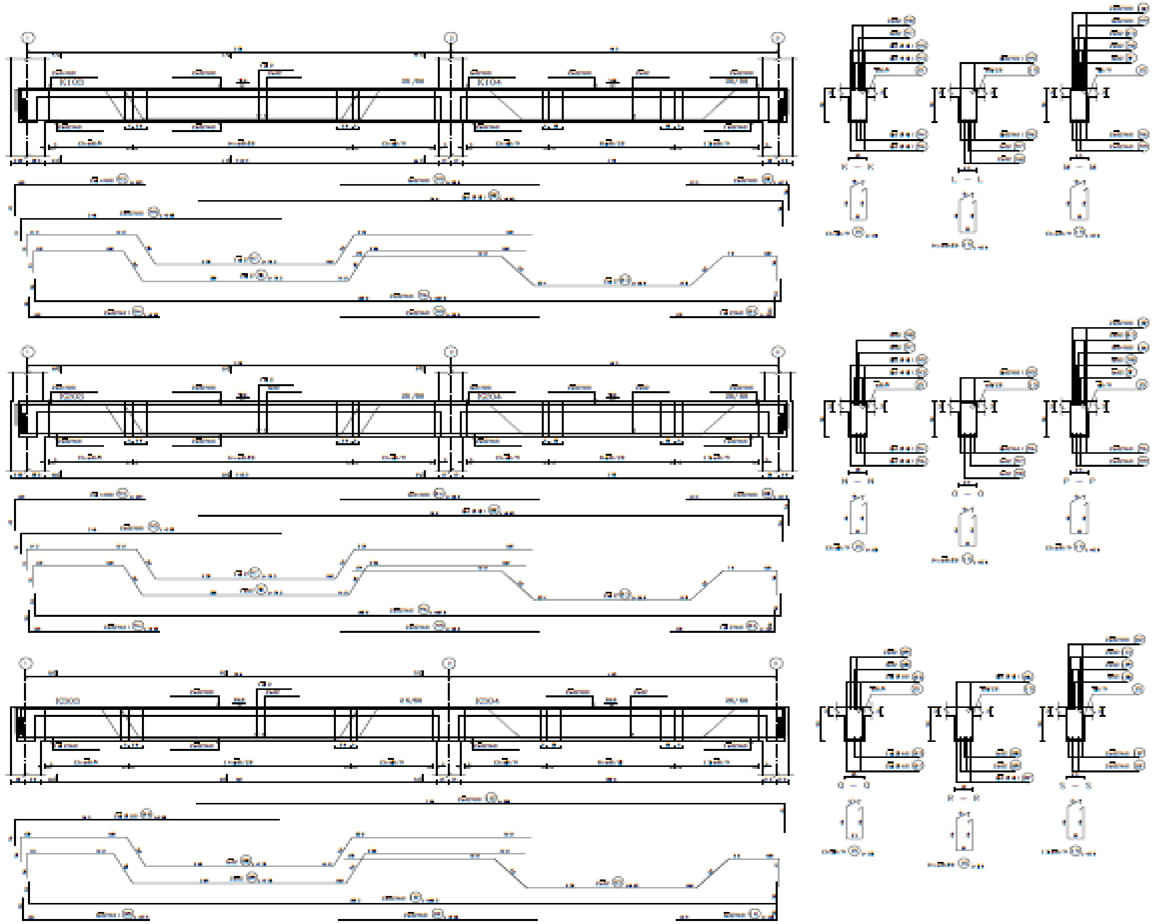


Figure A.7: Beam Details of 3-Story Frame

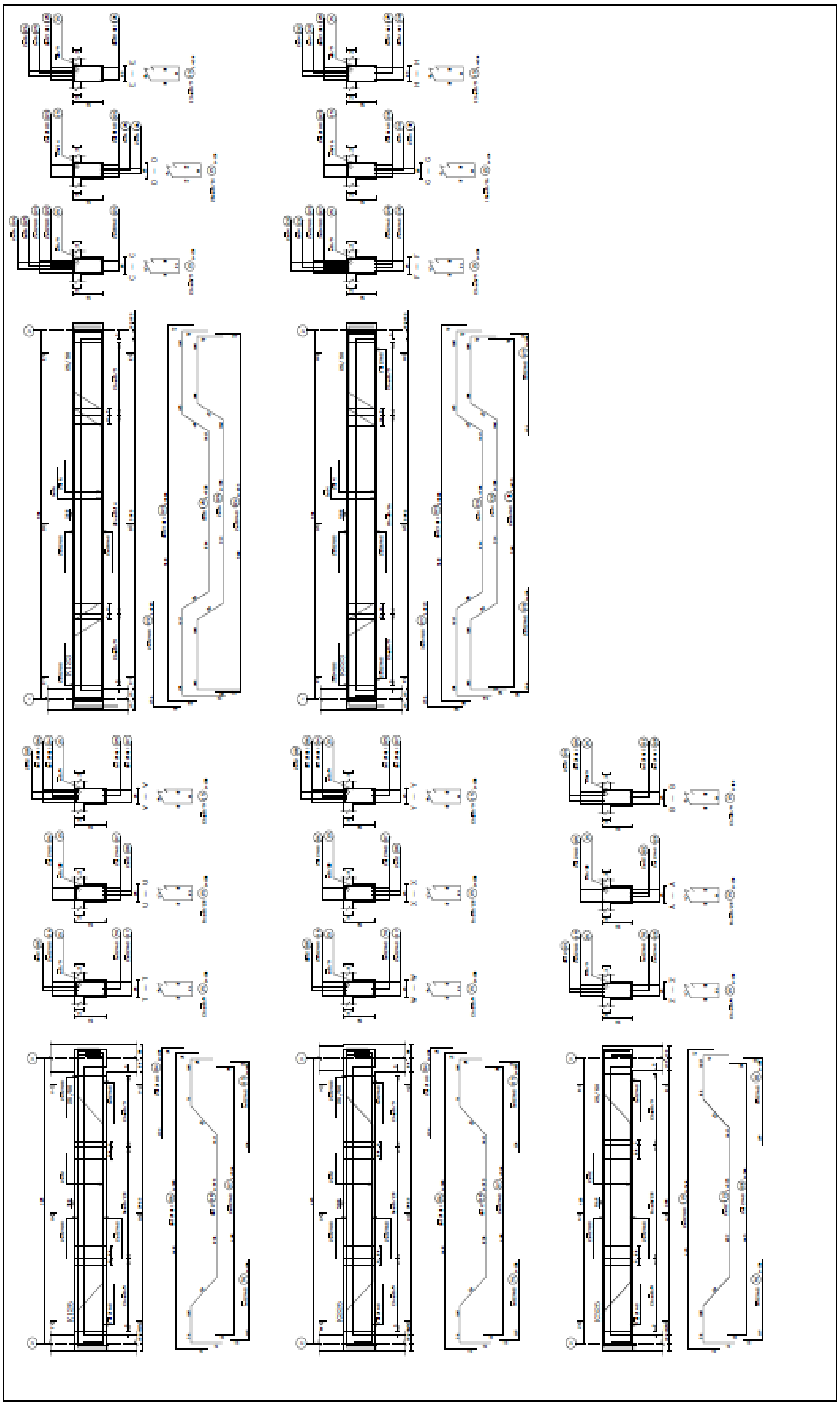
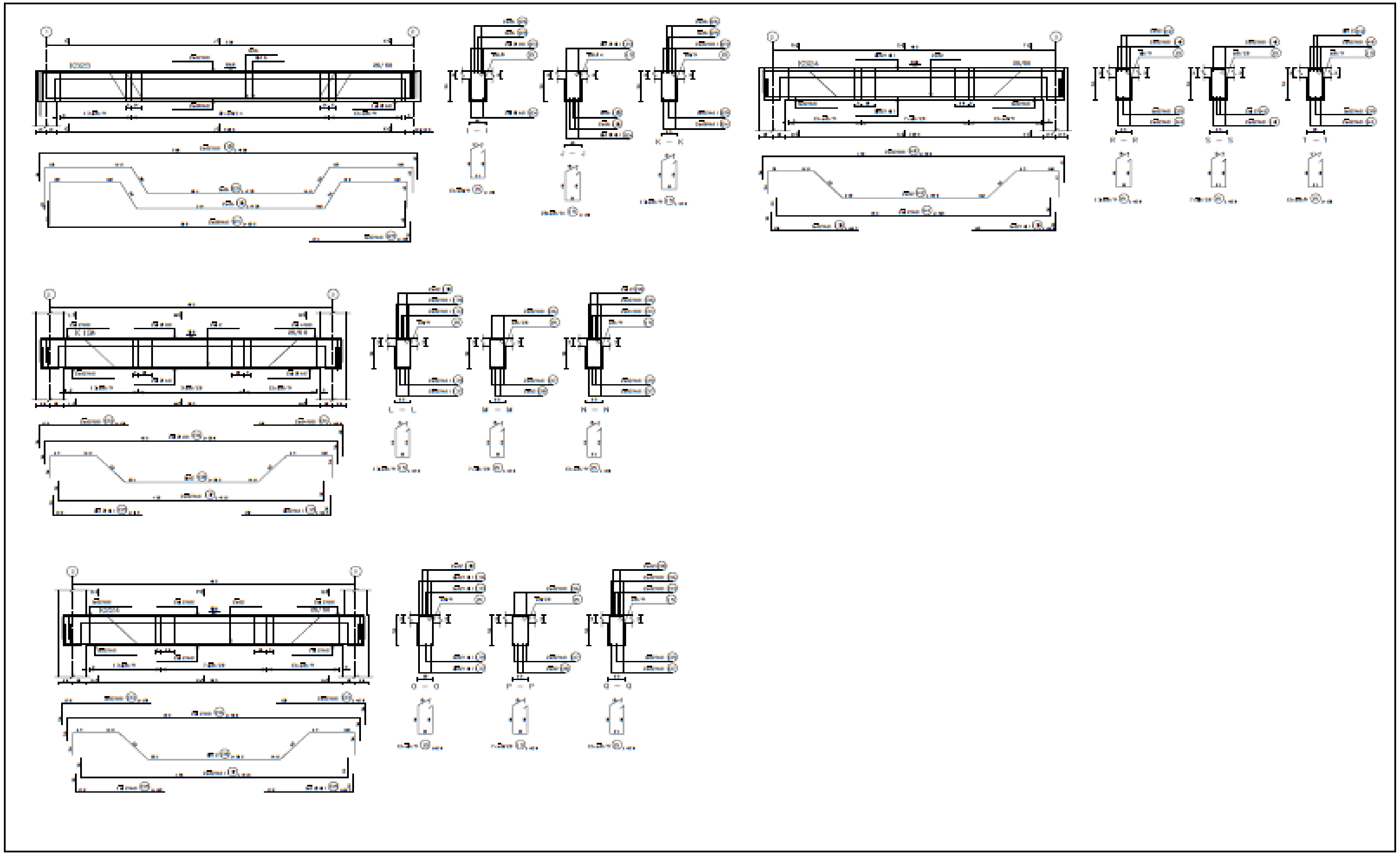


Figure A.8: Beam Details of 3-Story Frame



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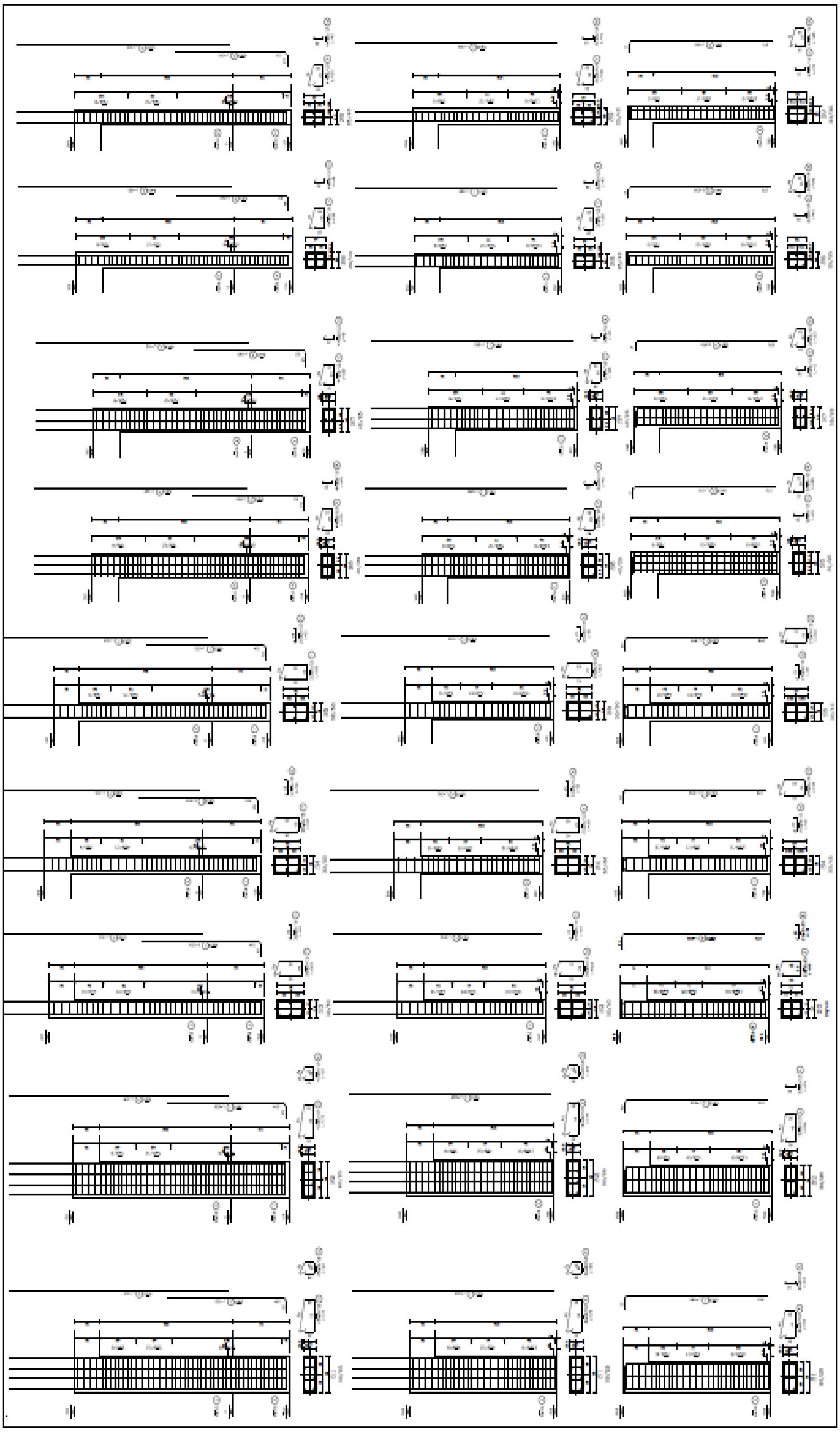

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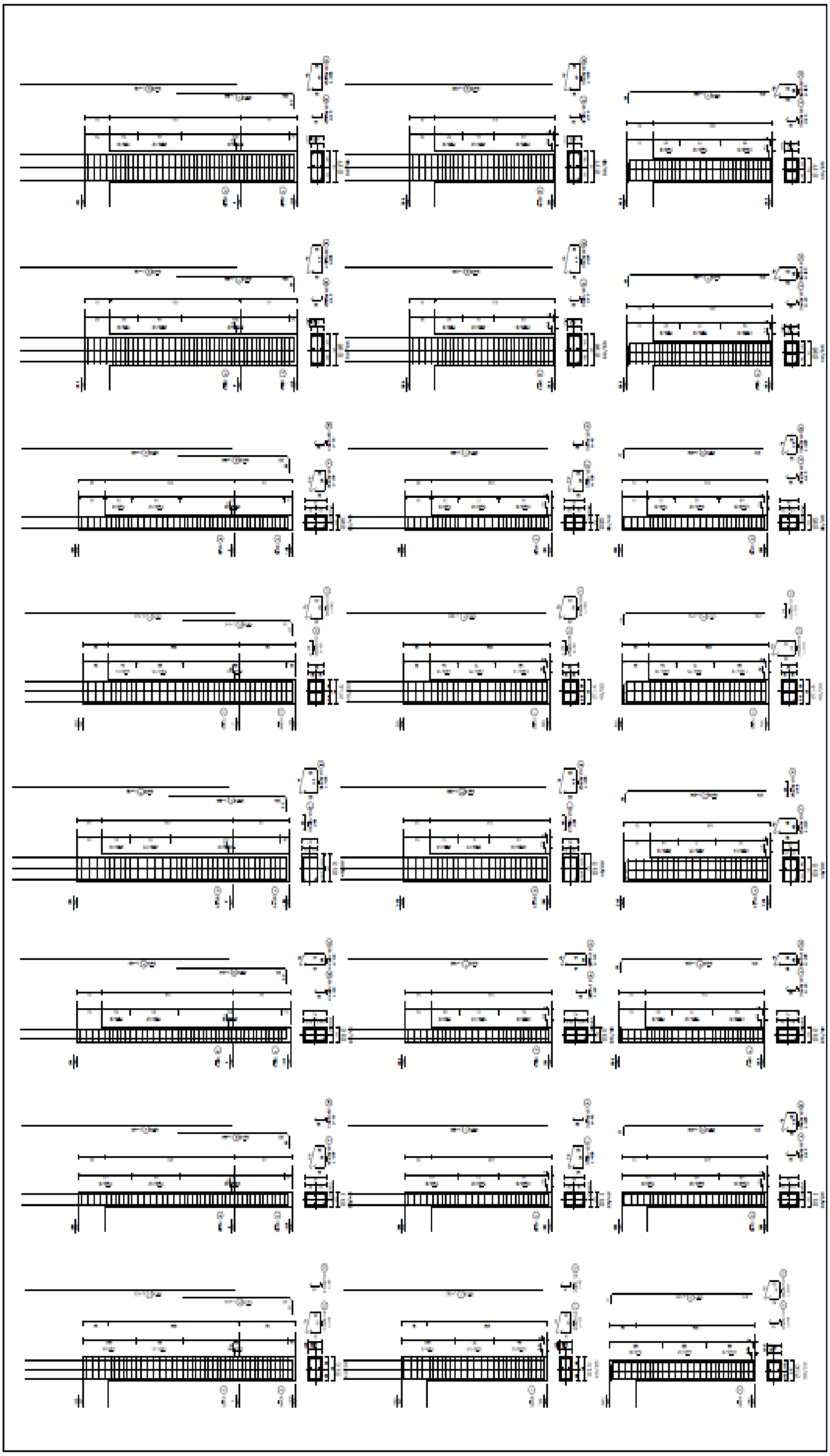

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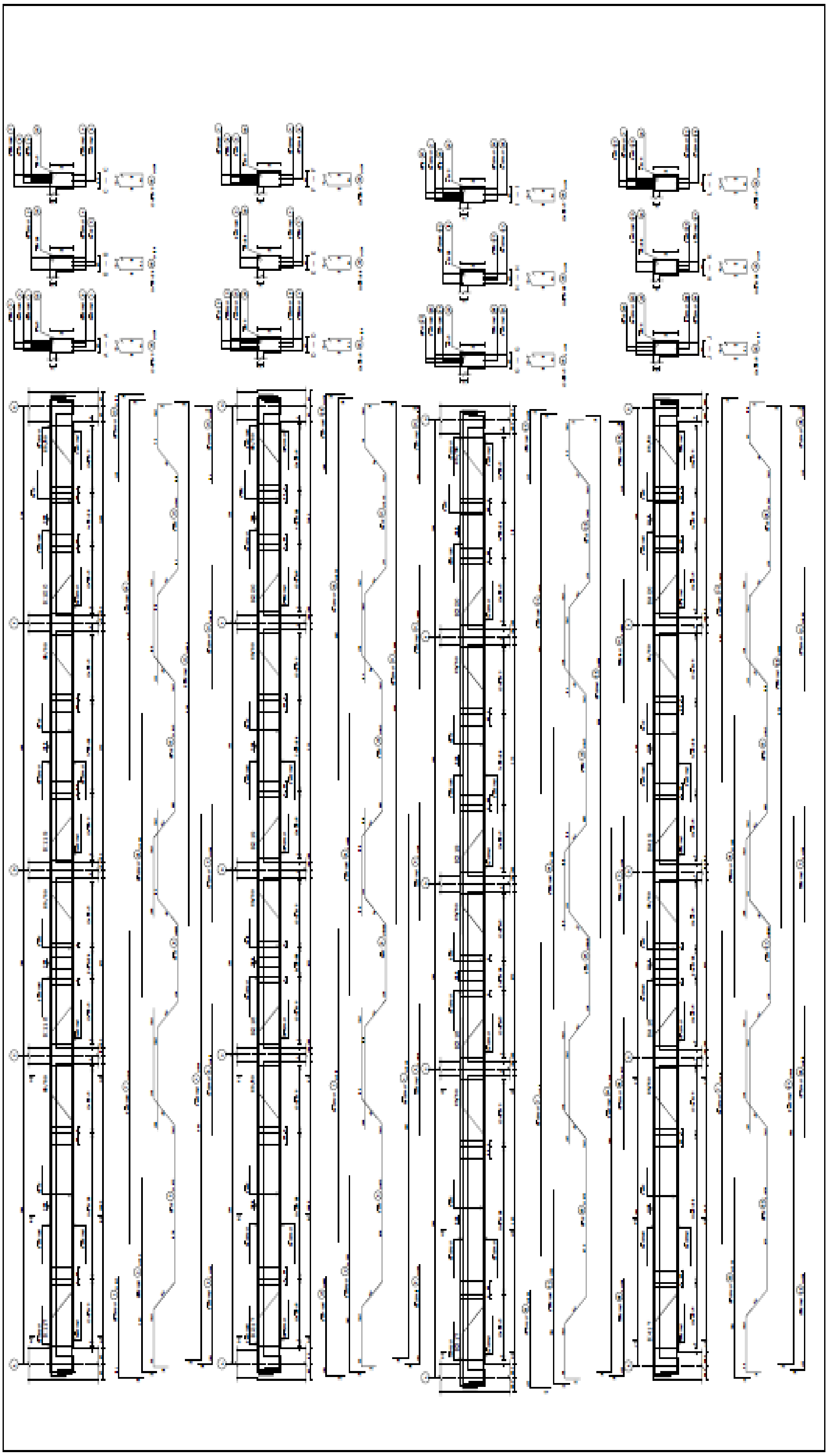

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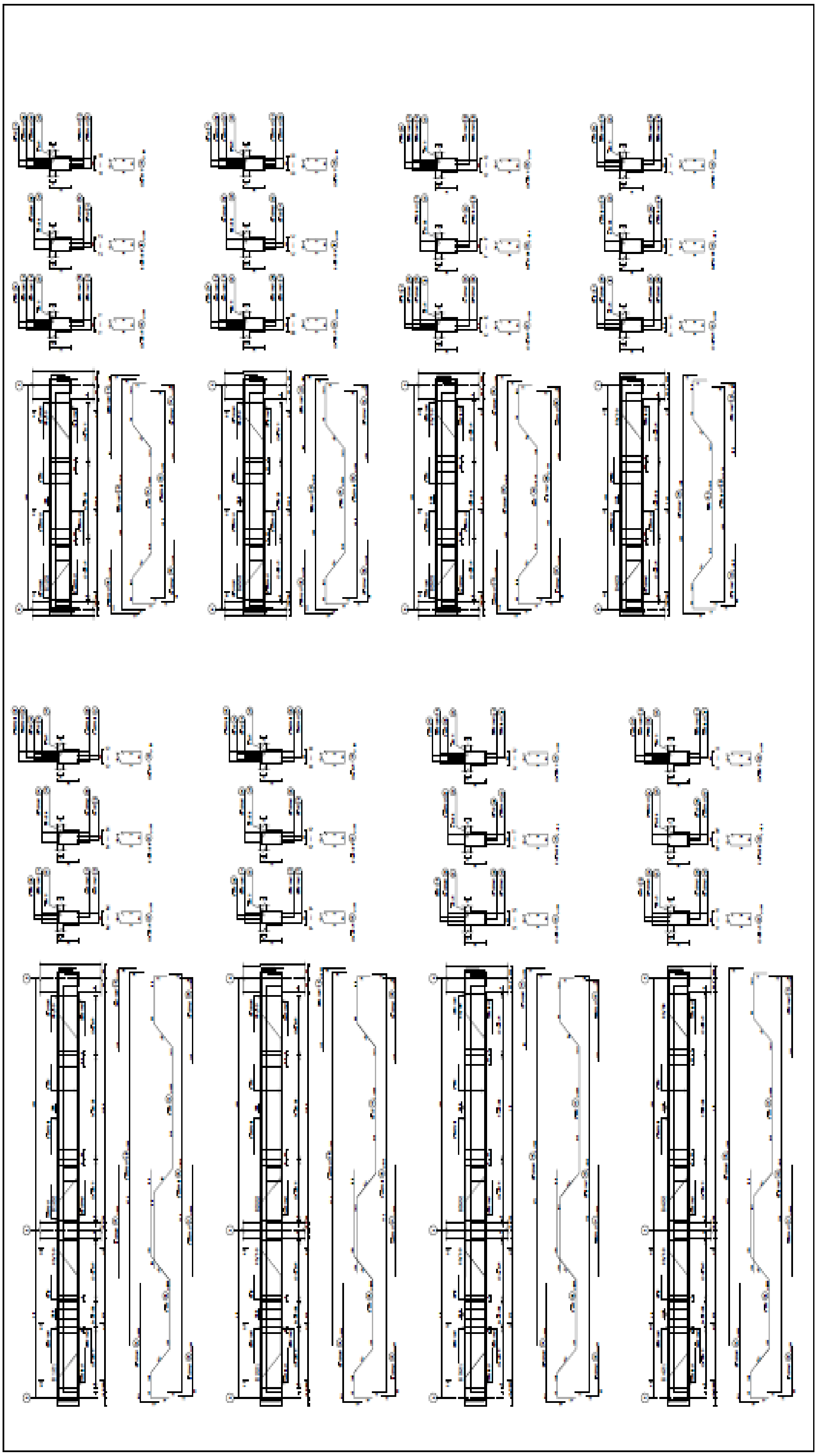

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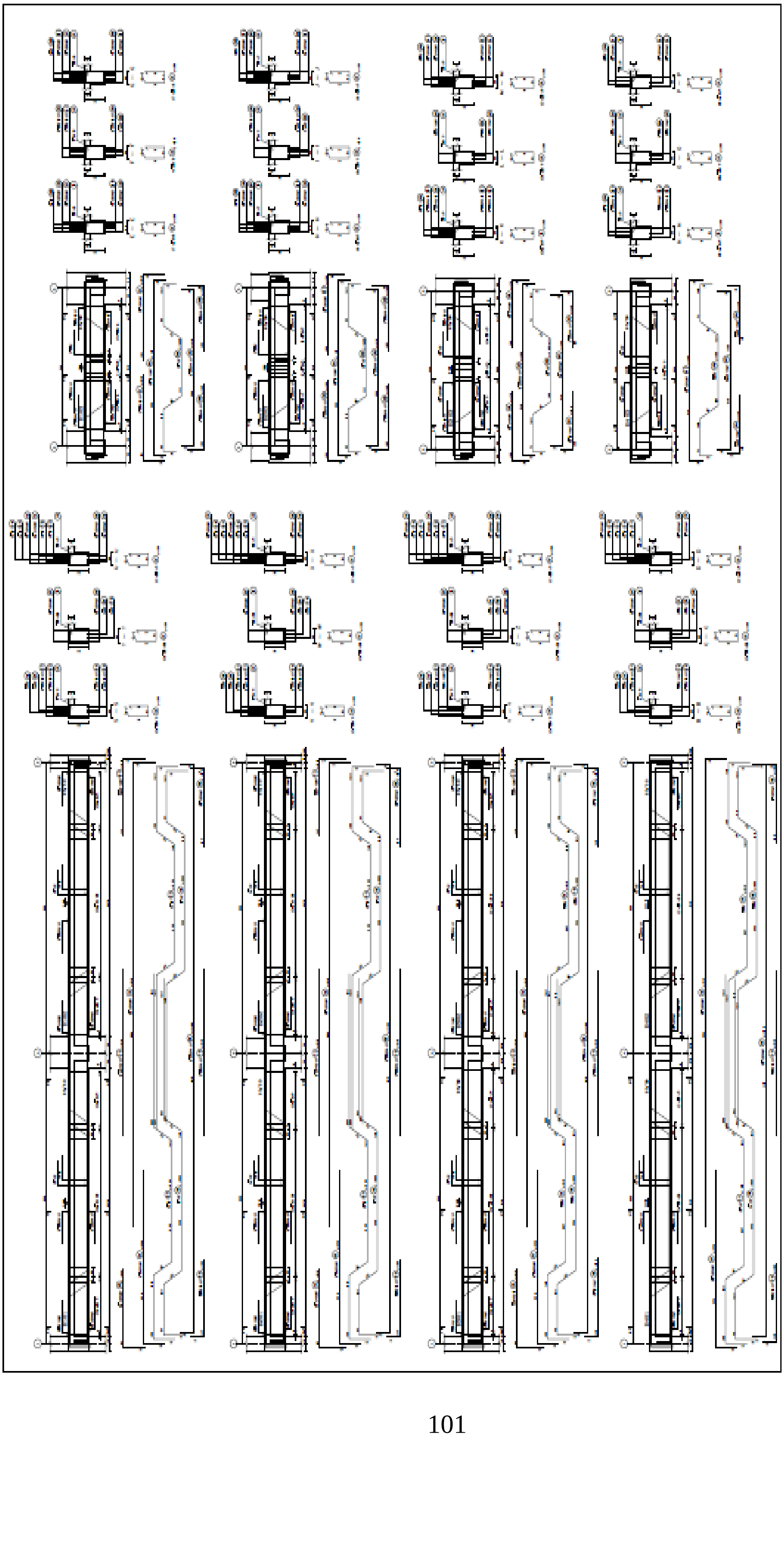

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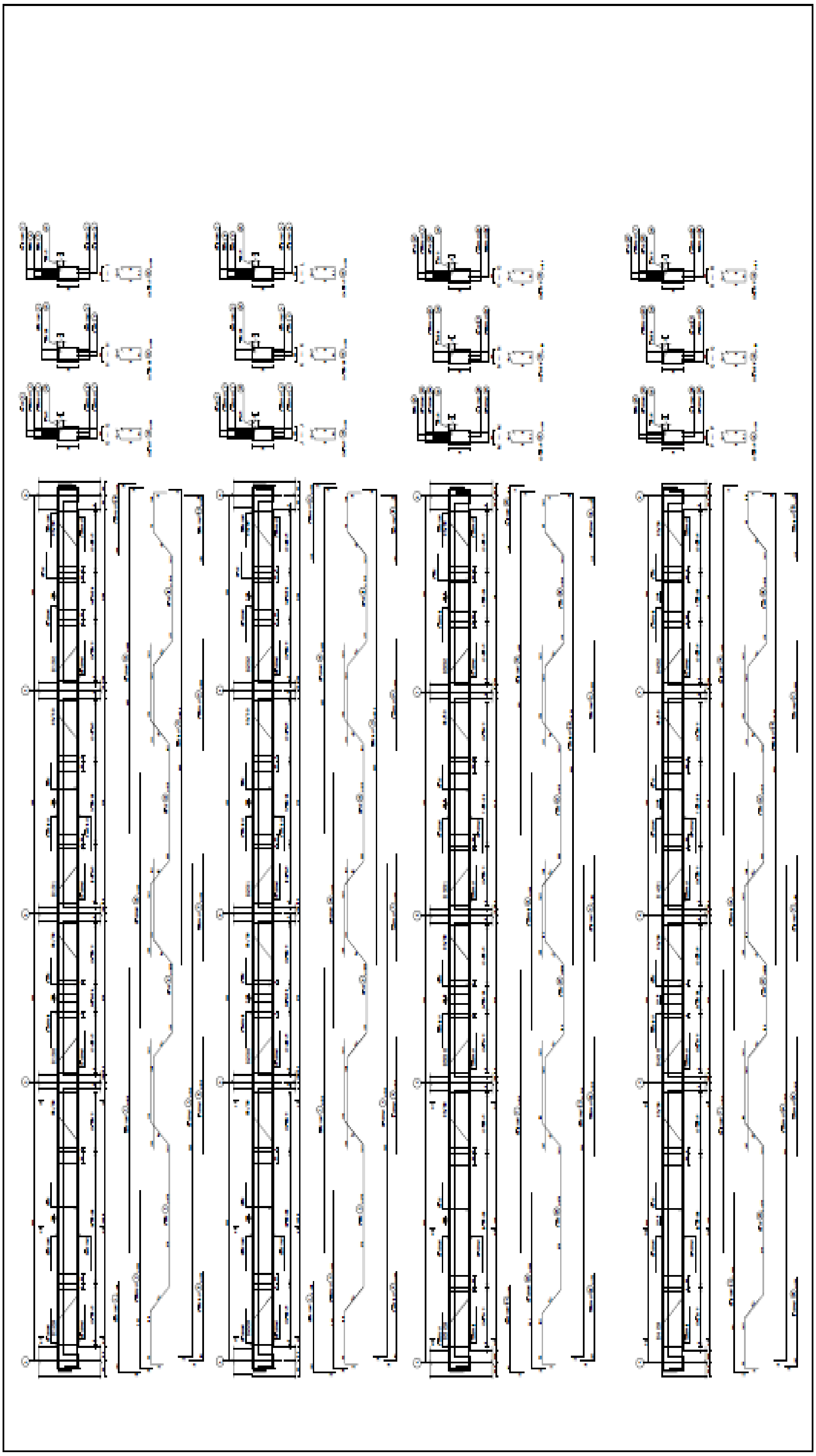

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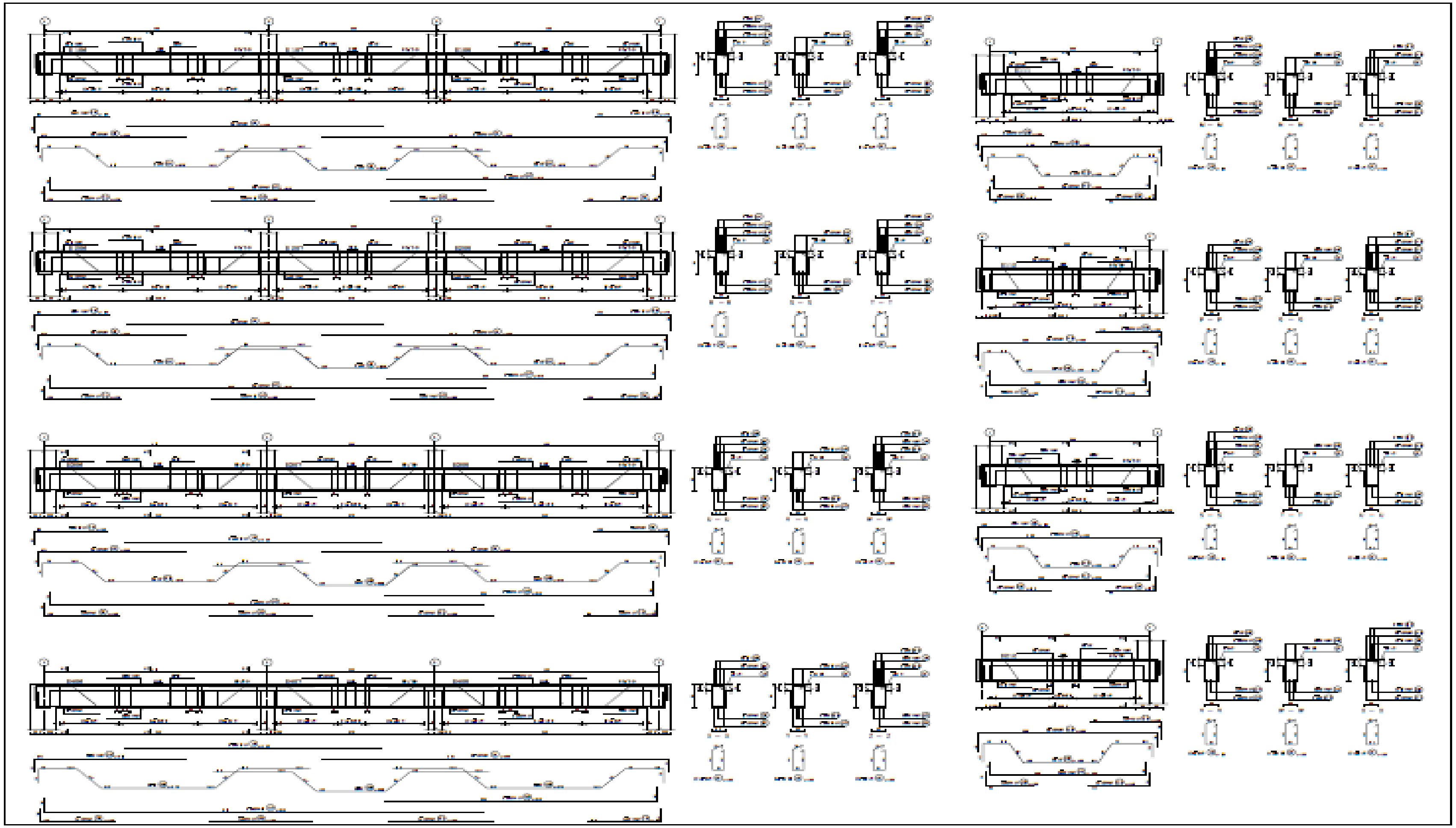

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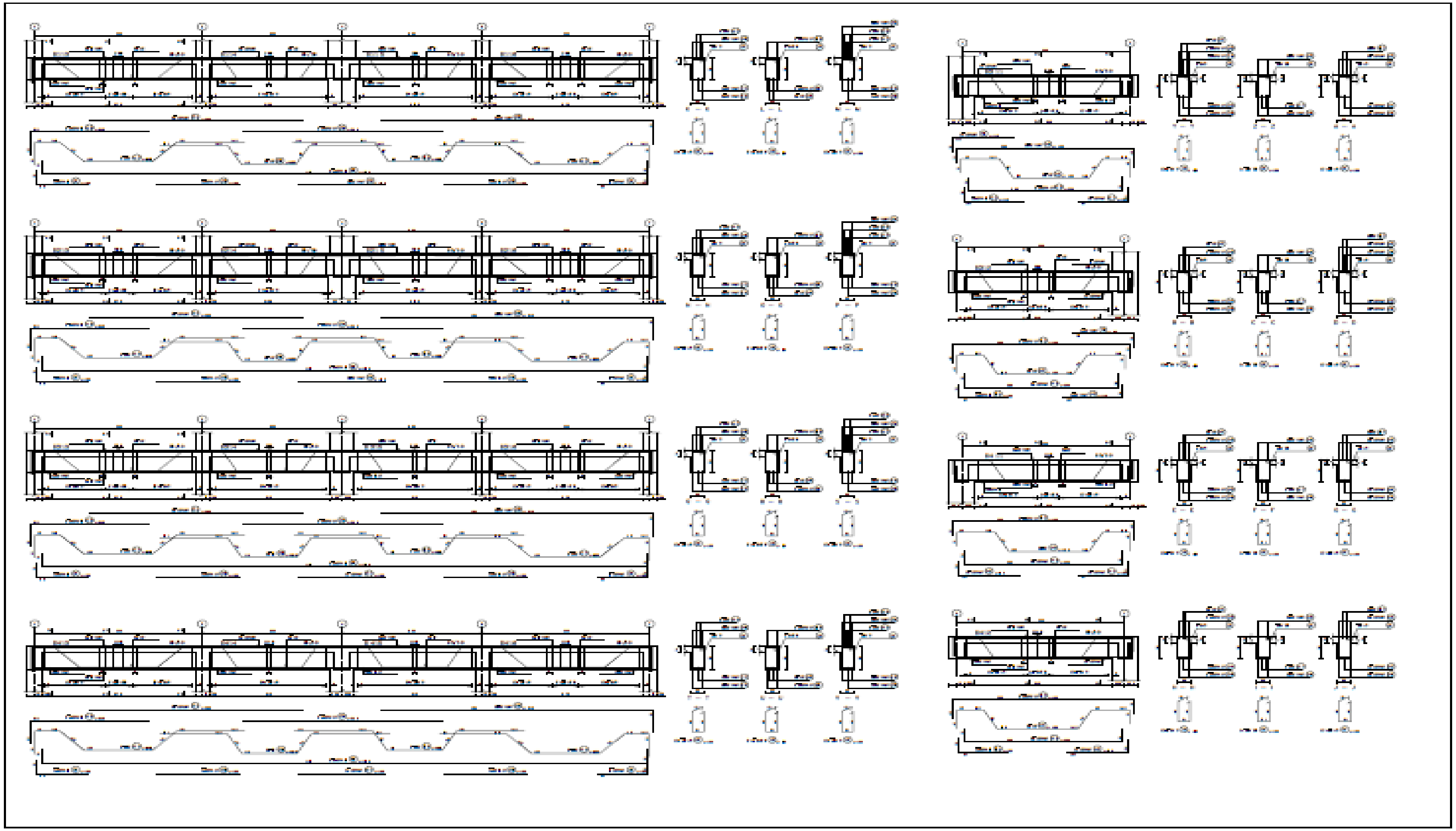

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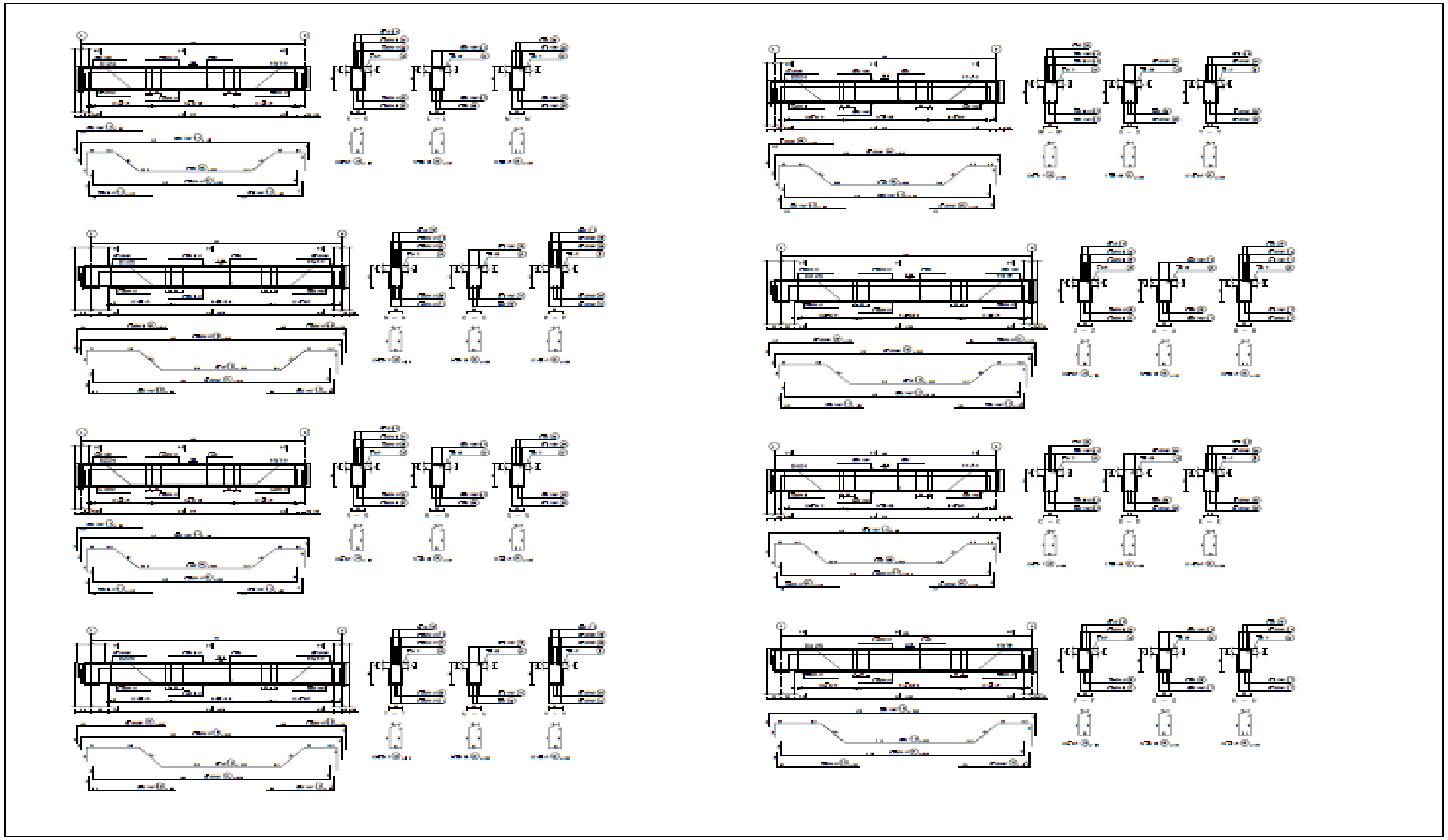

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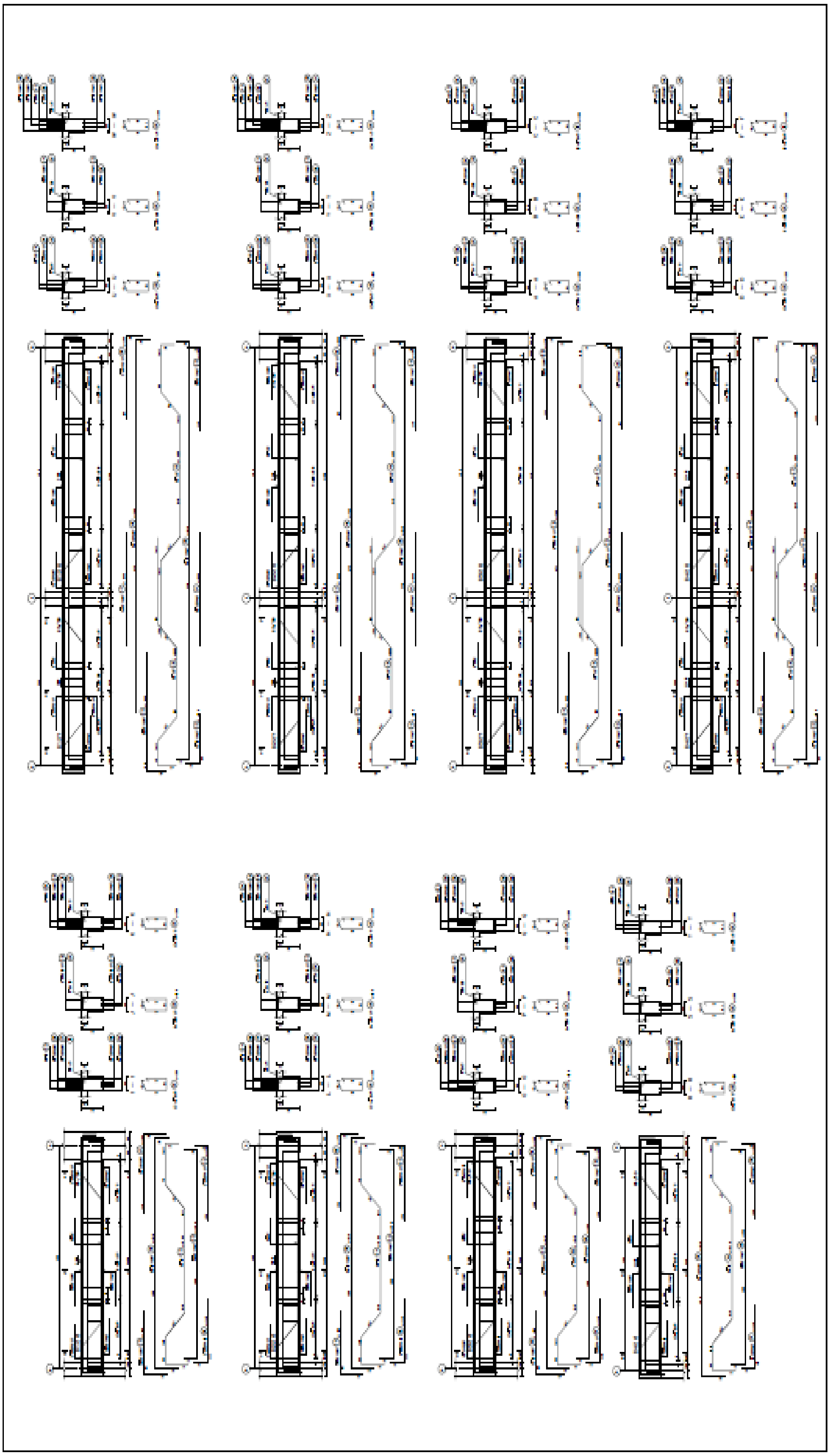

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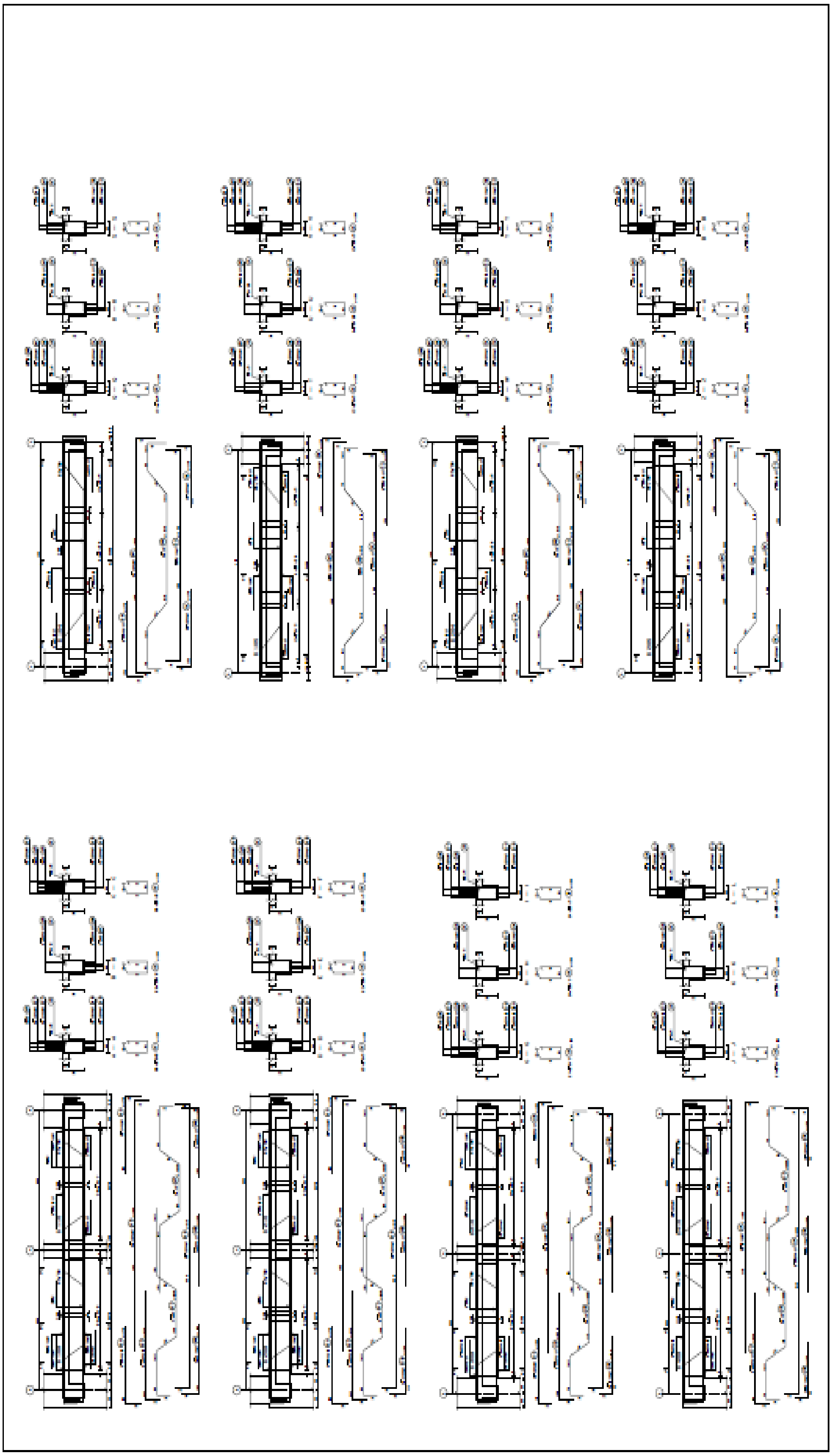

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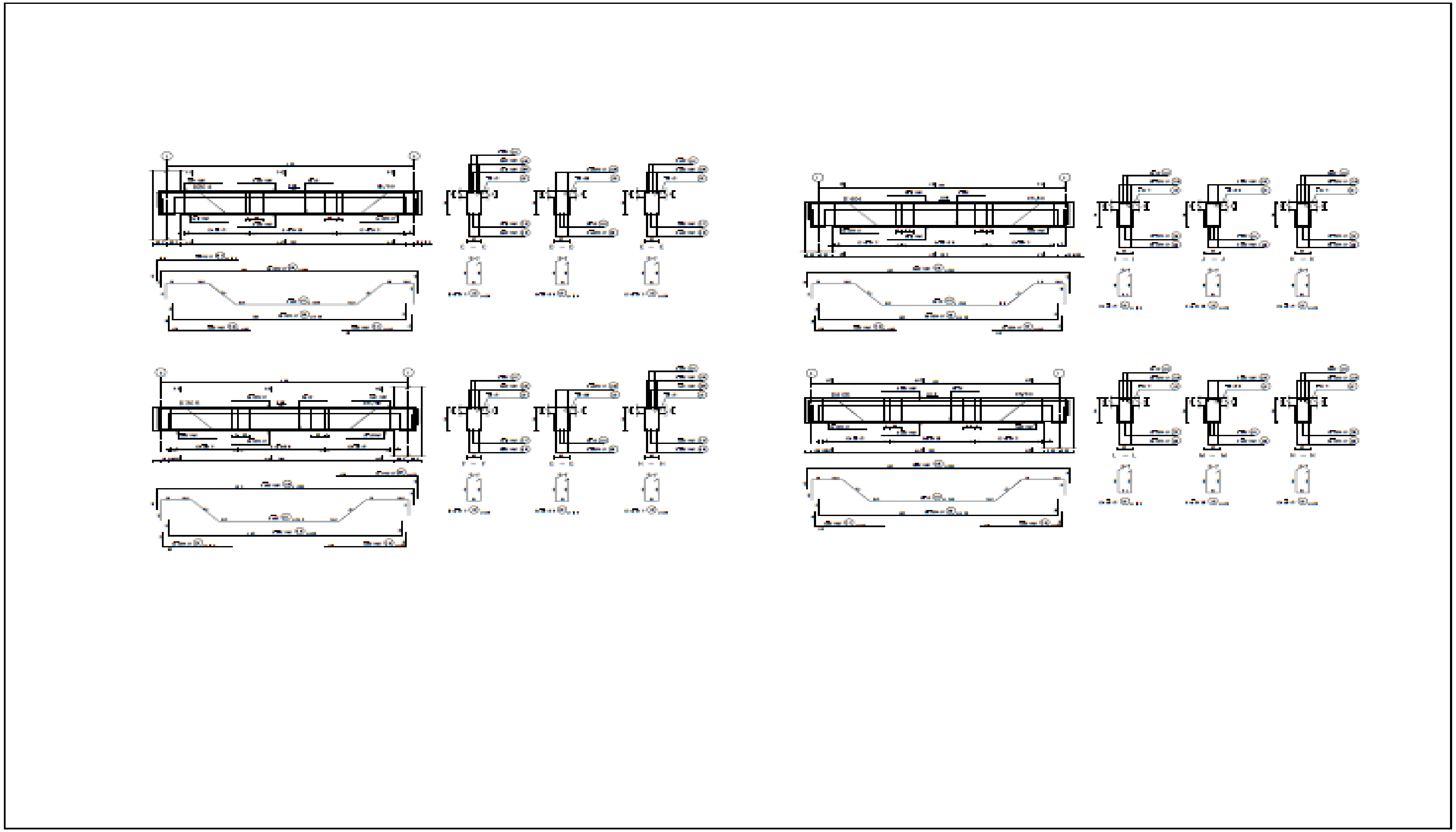

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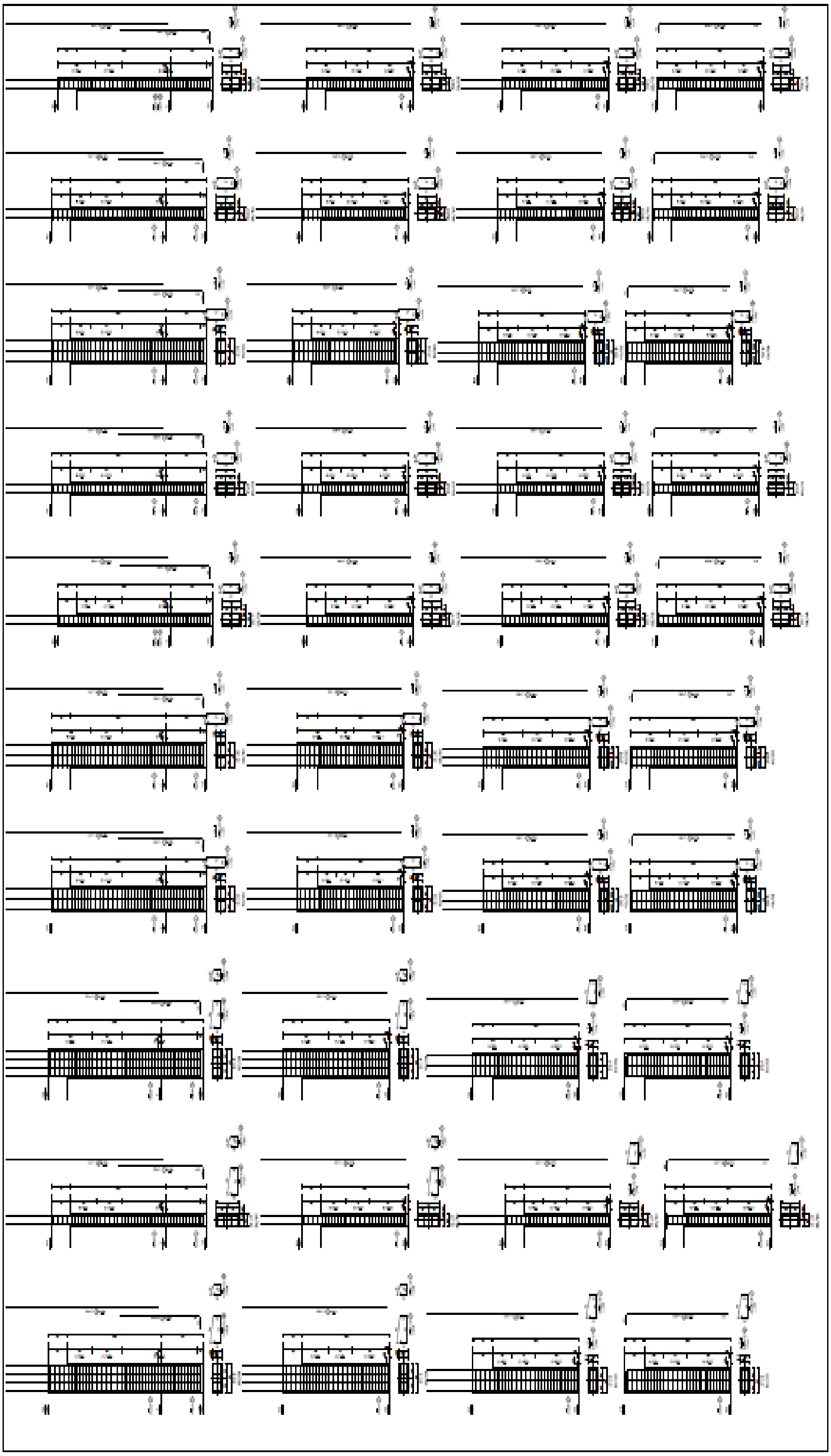

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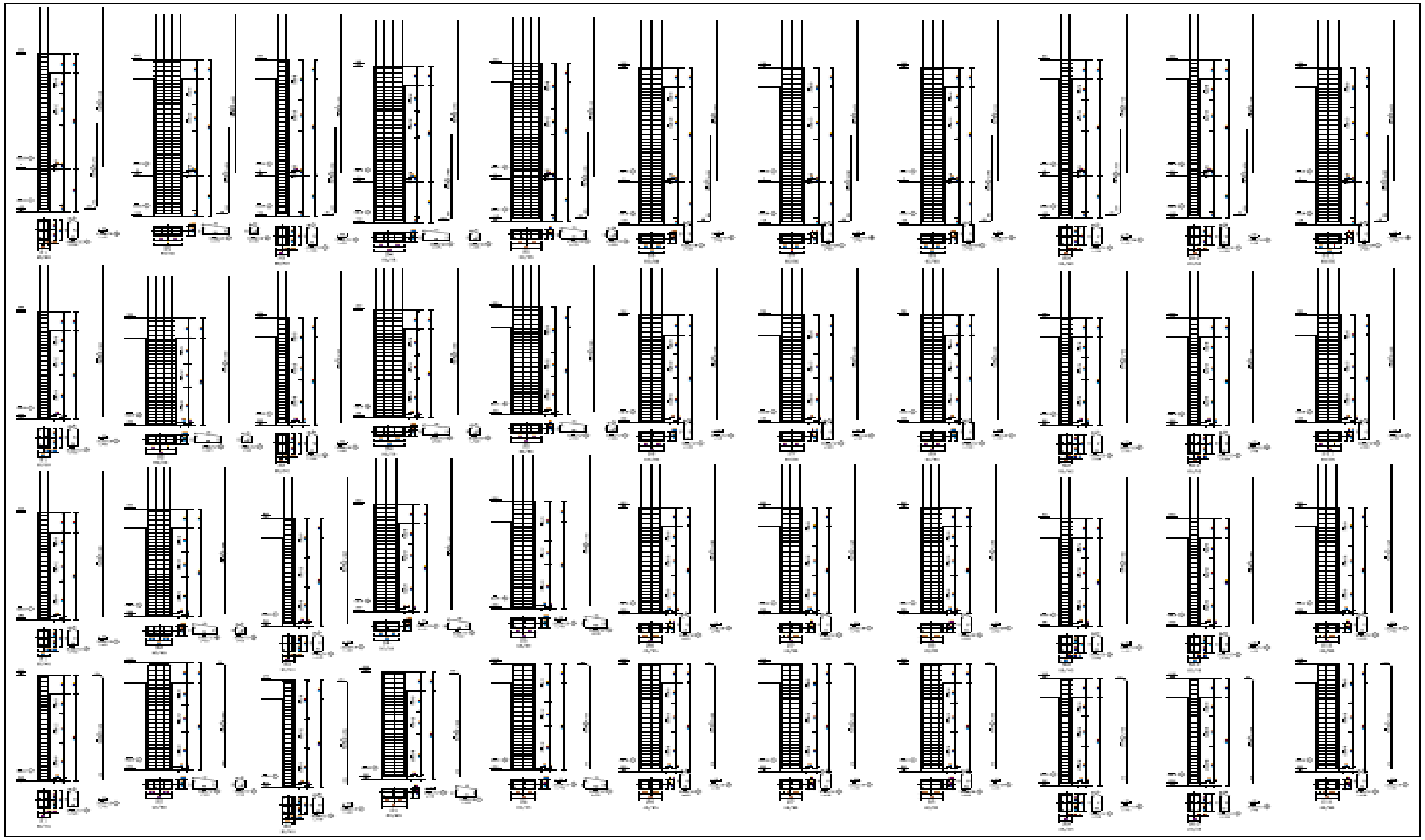

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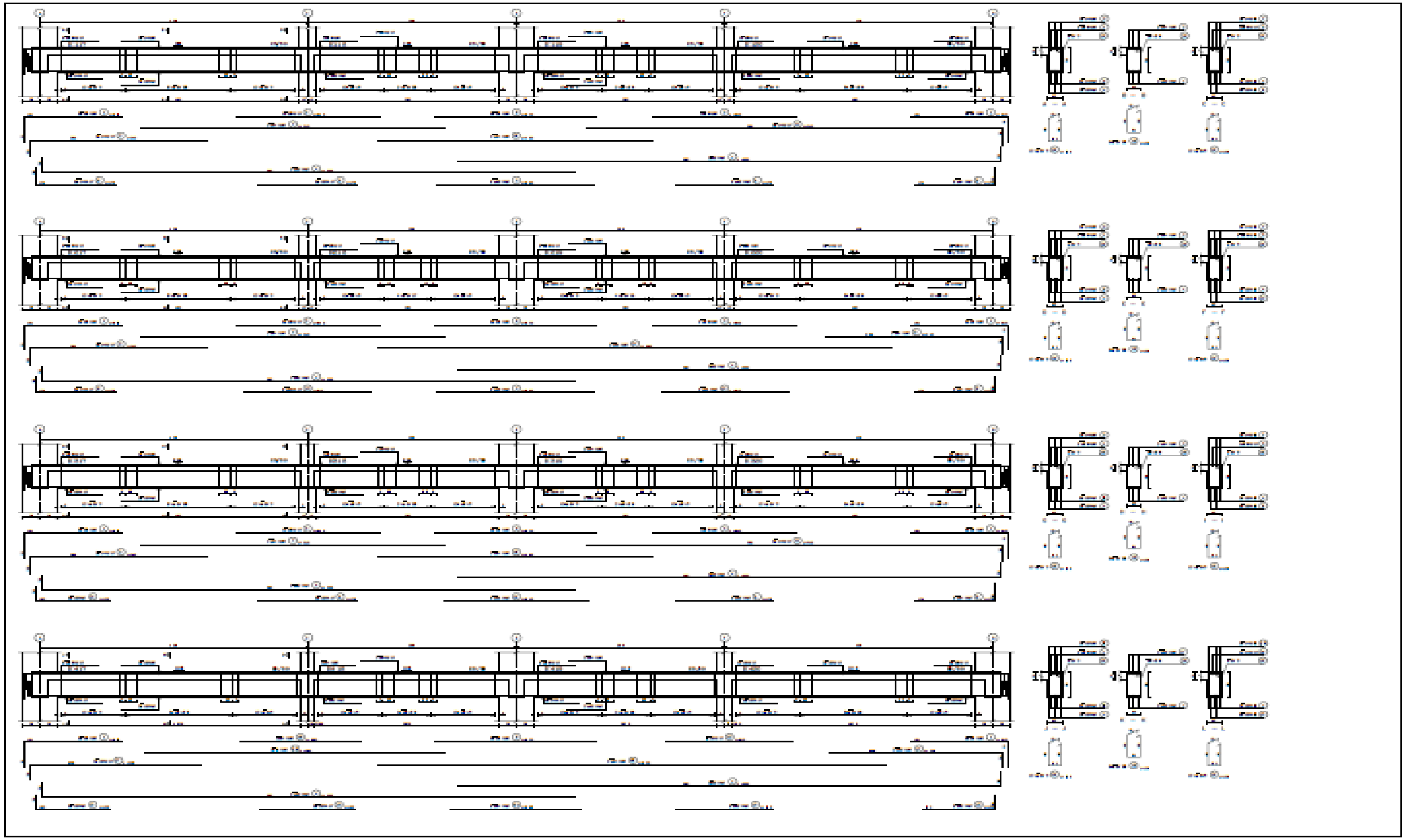

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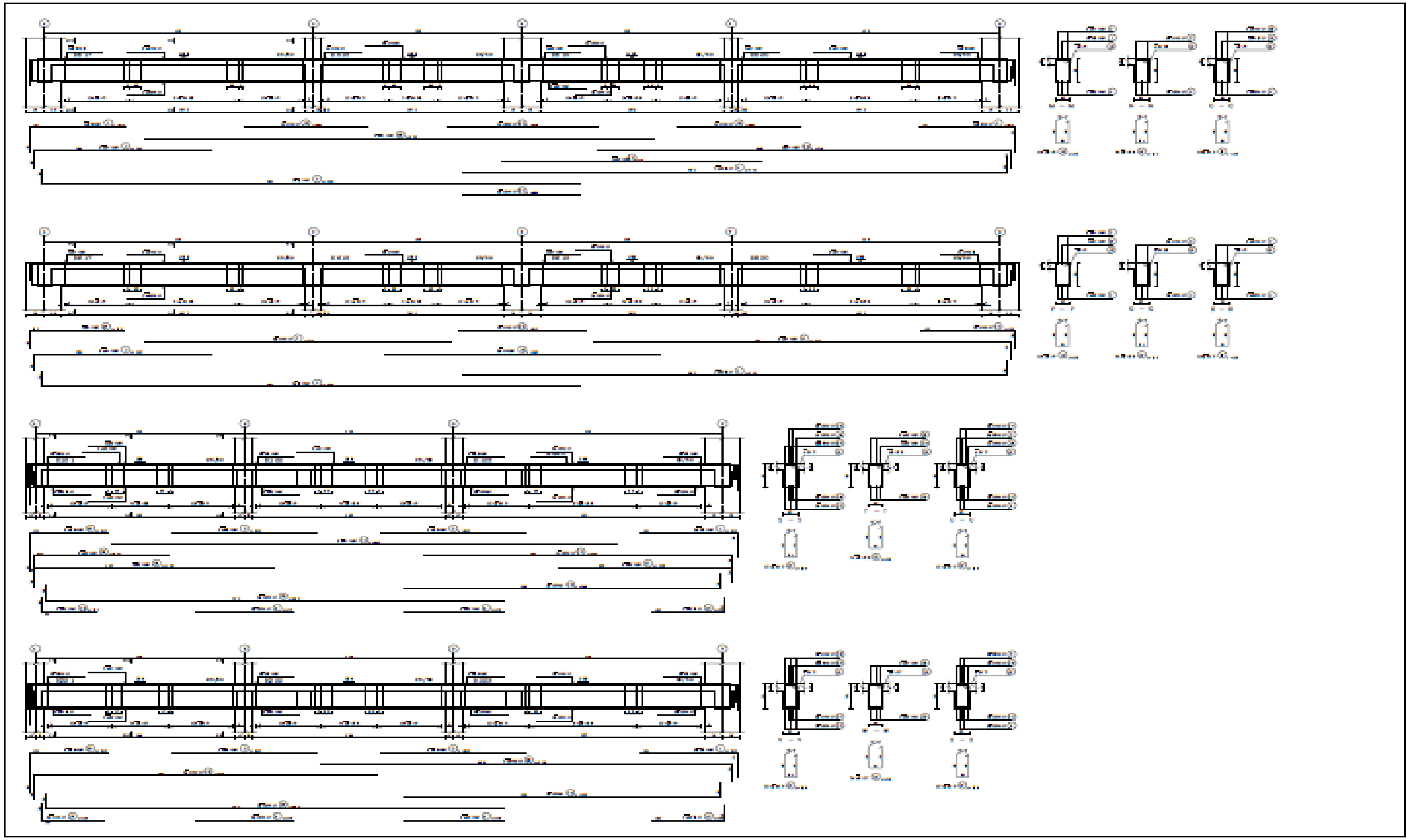

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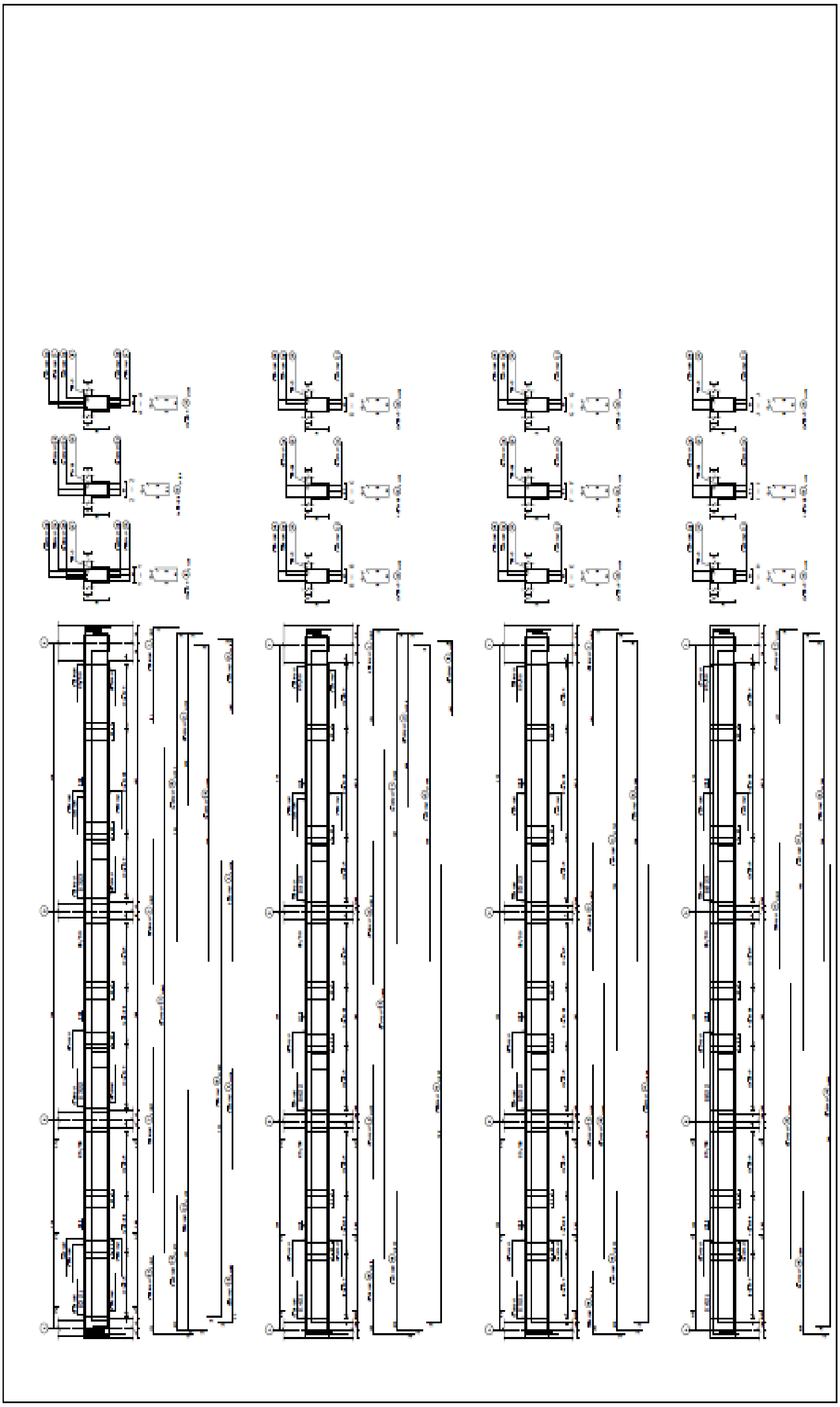

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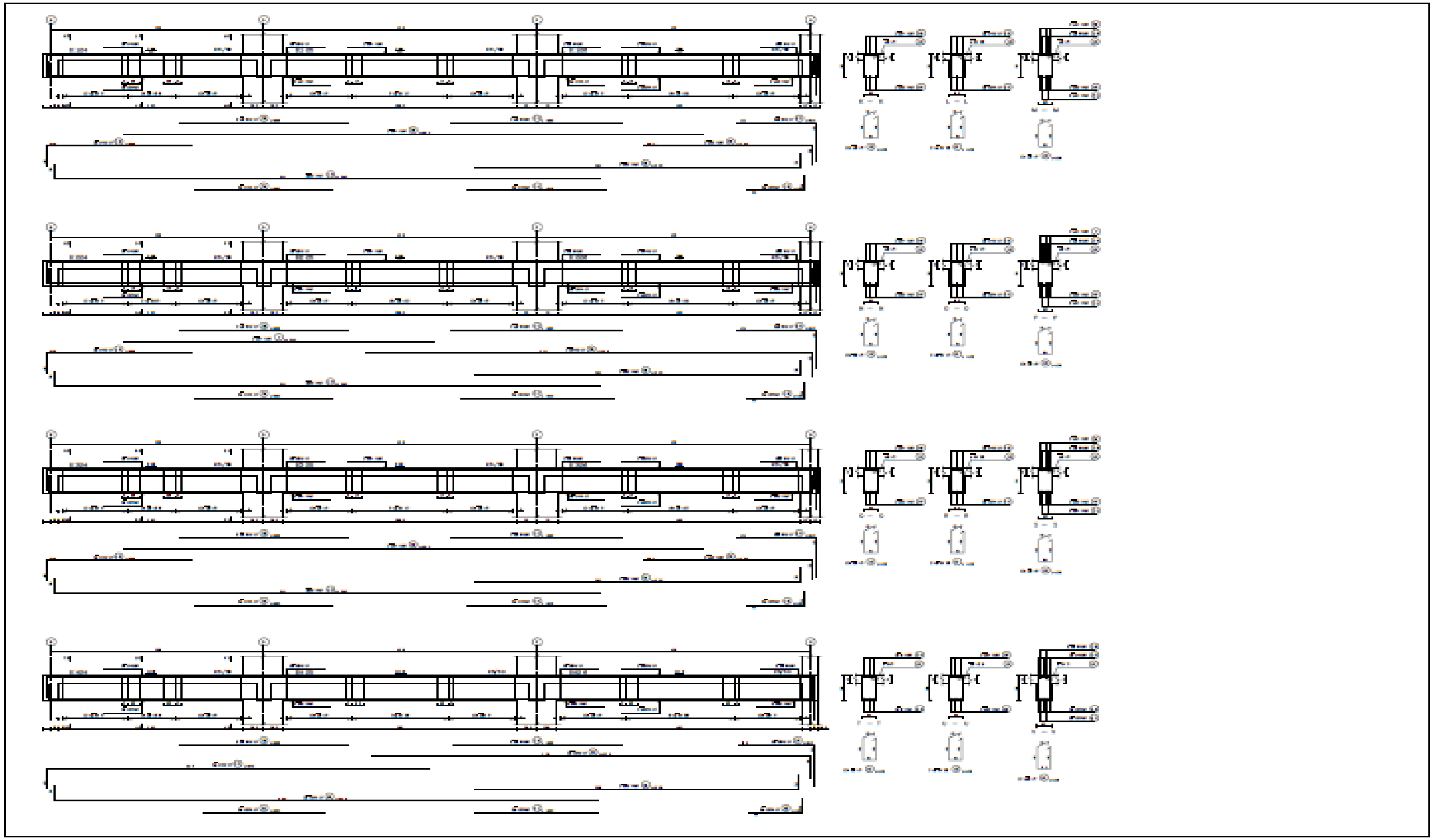

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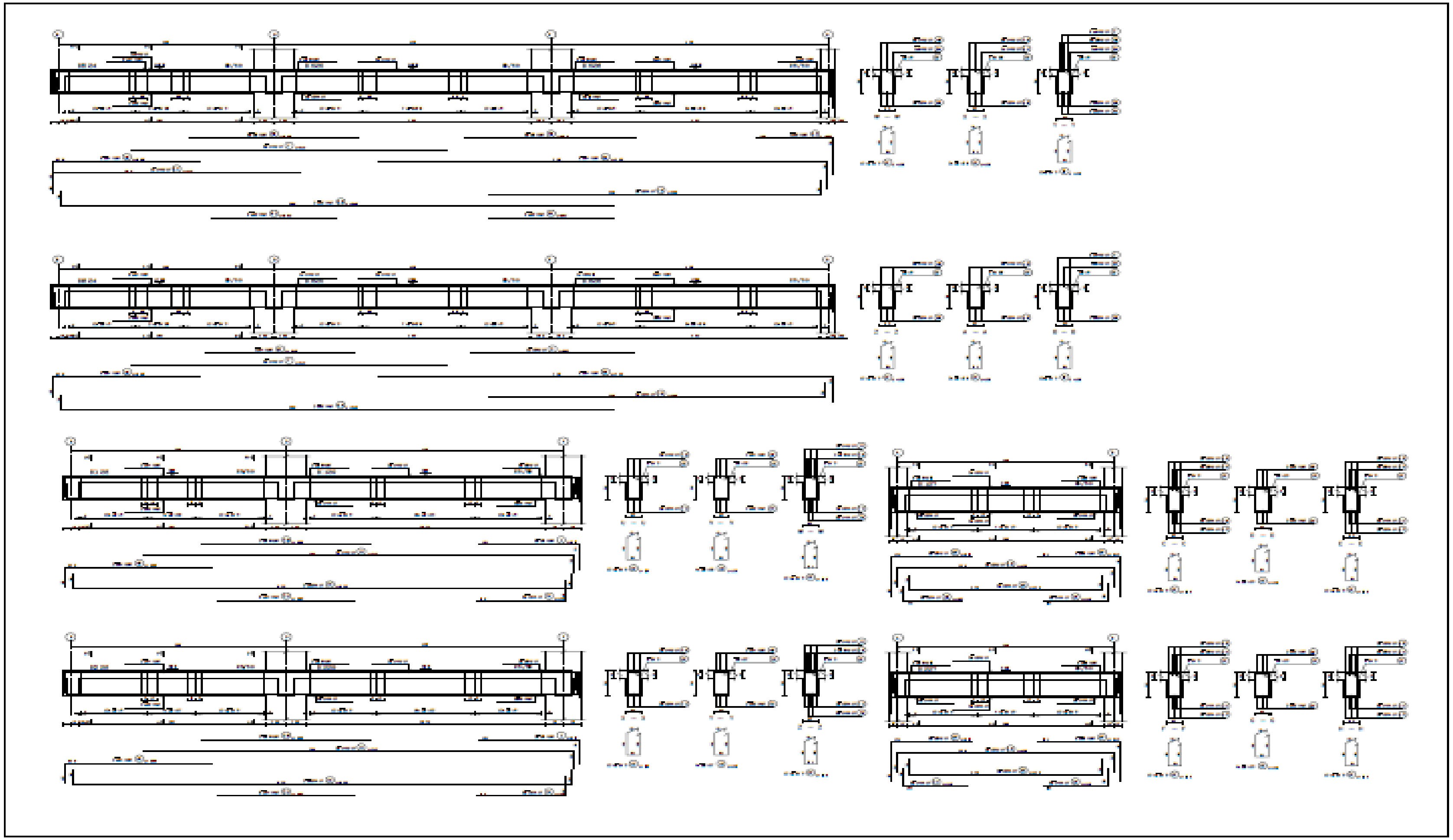

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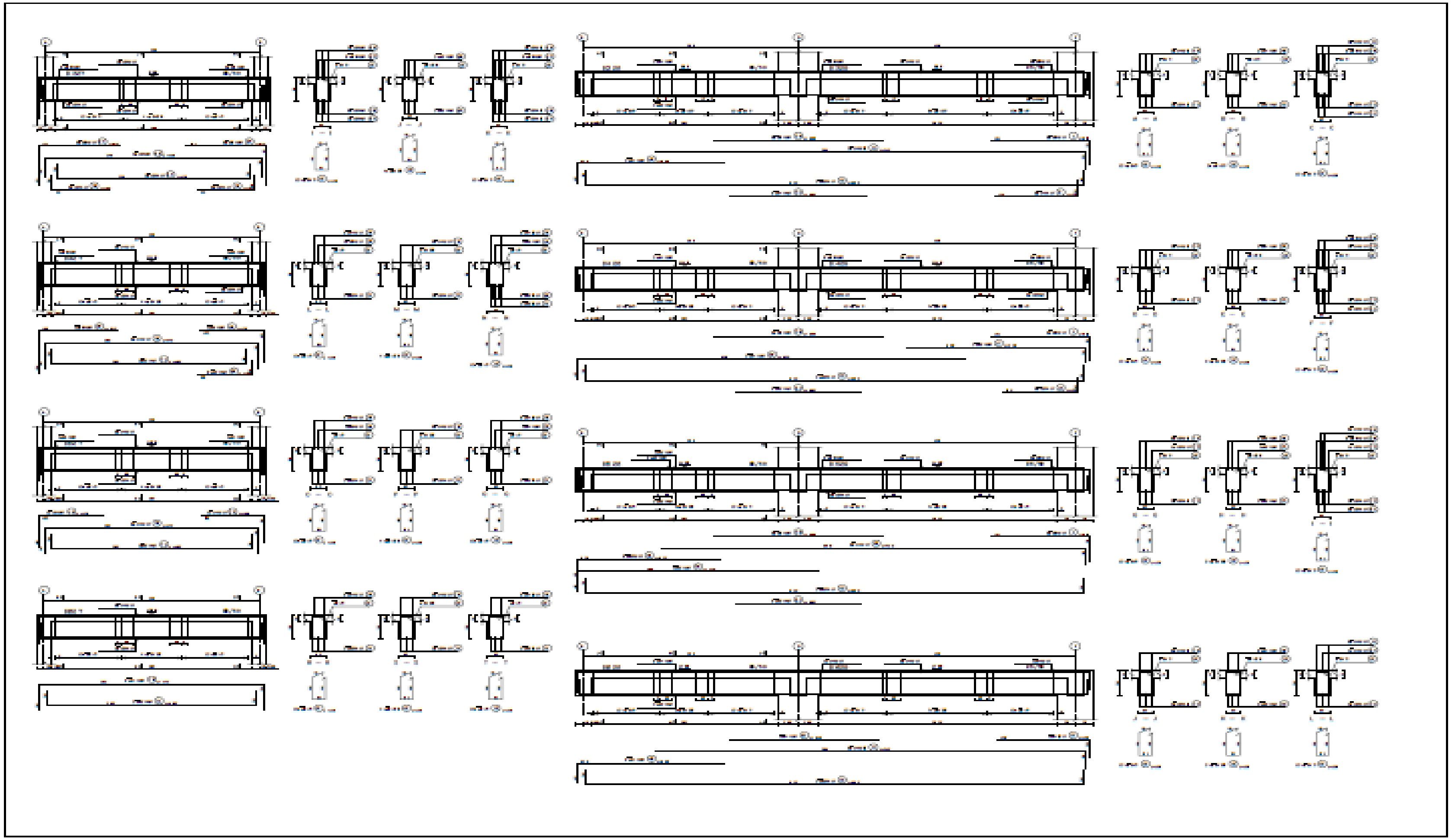

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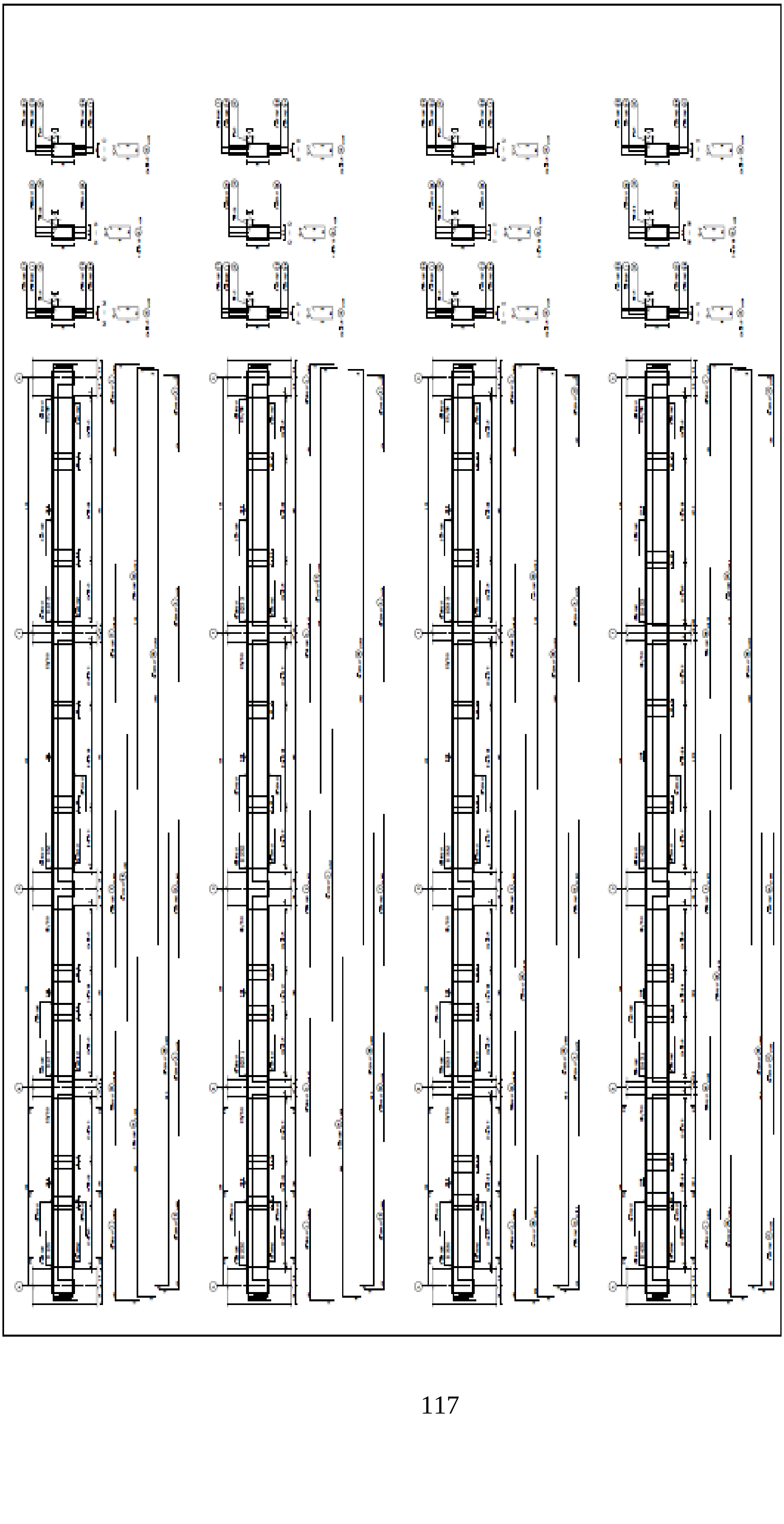

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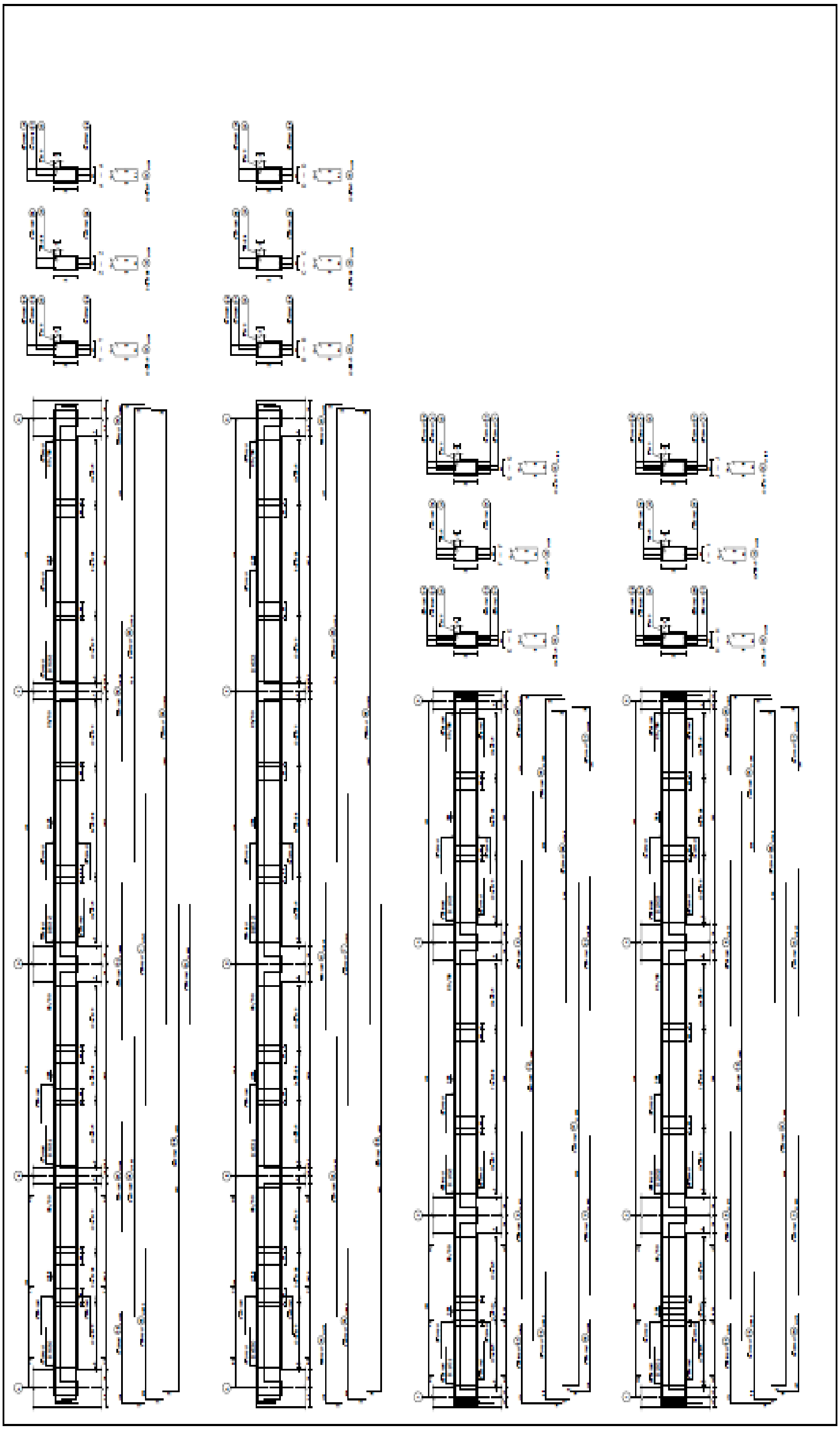

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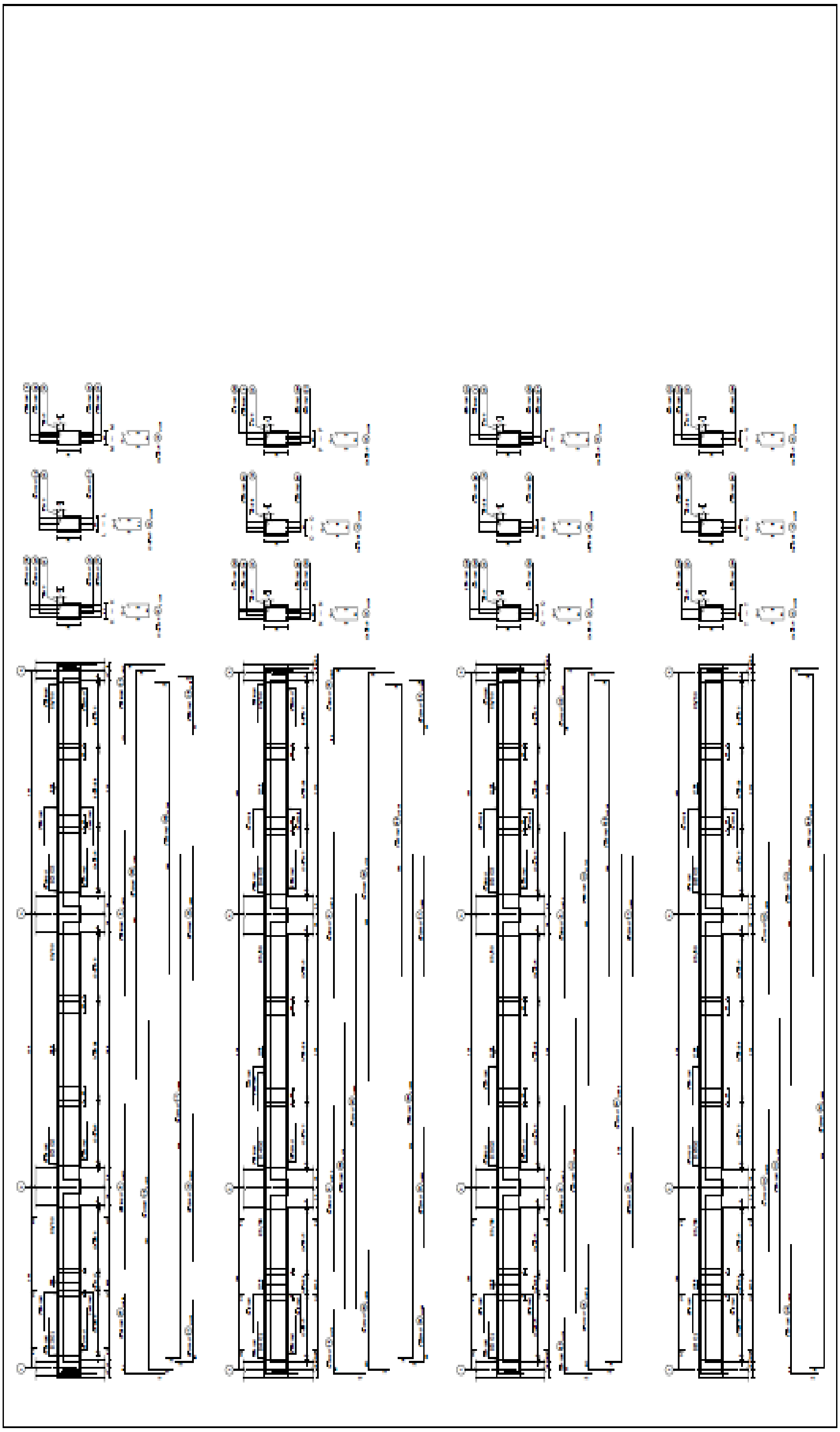

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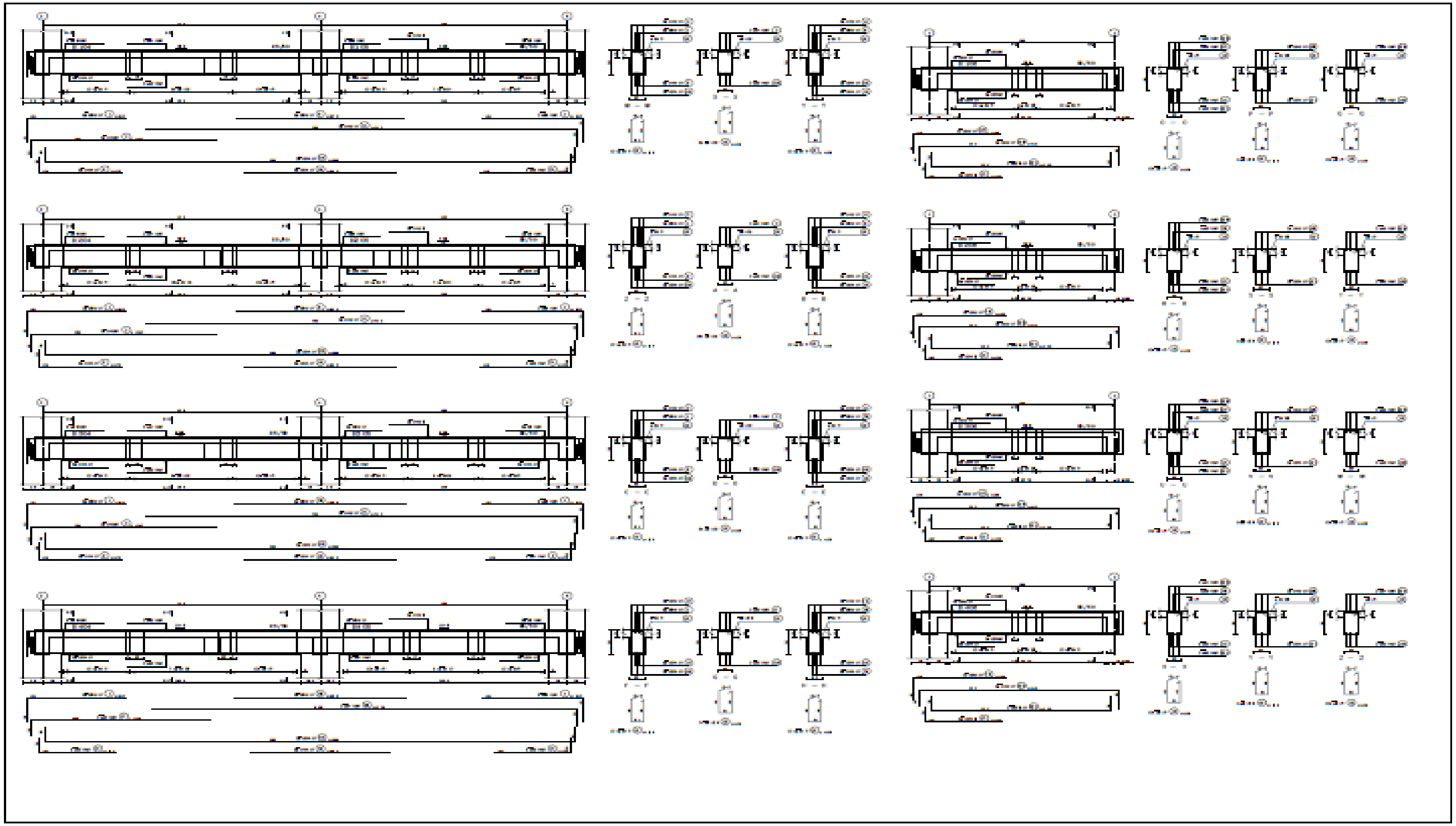

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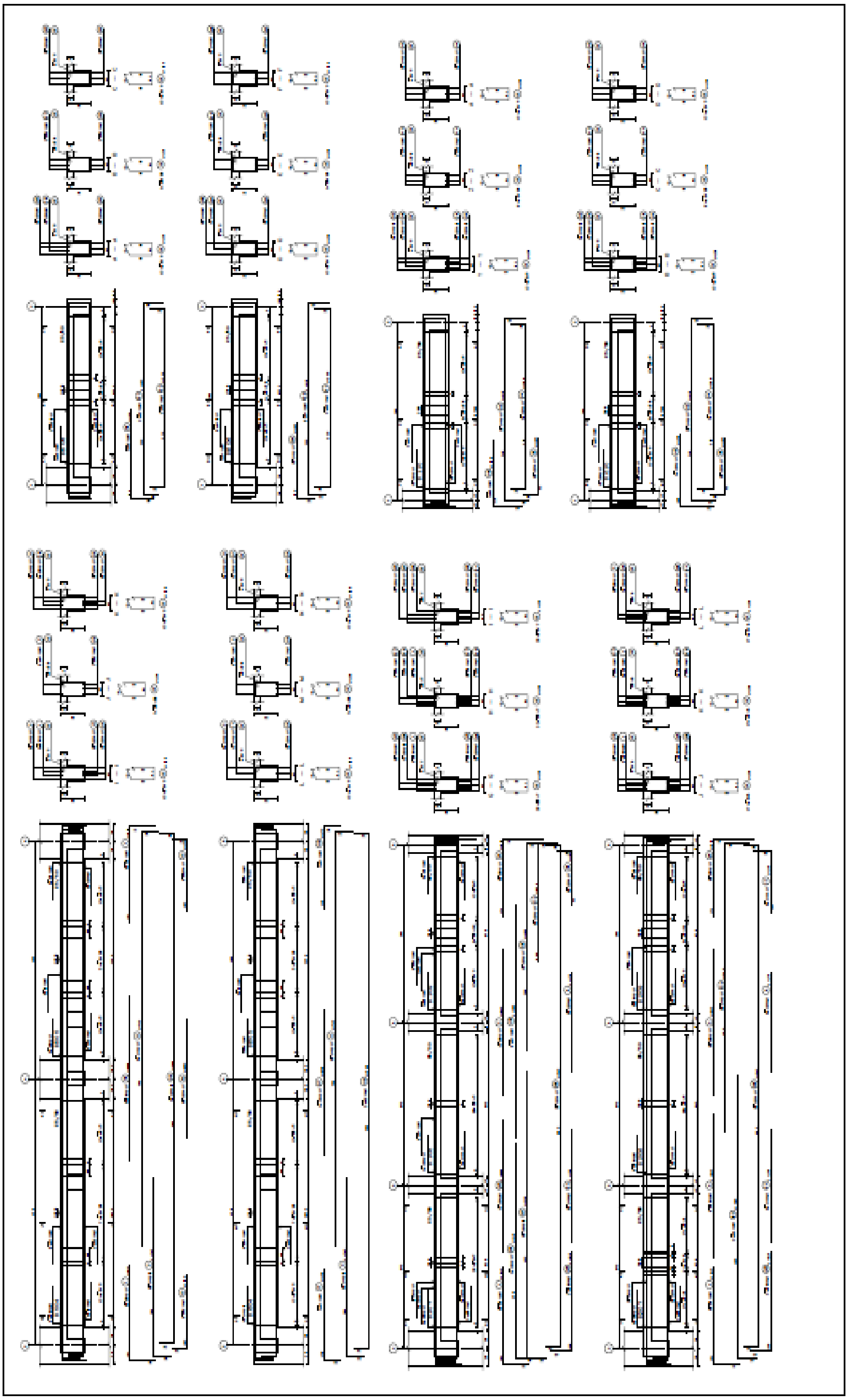

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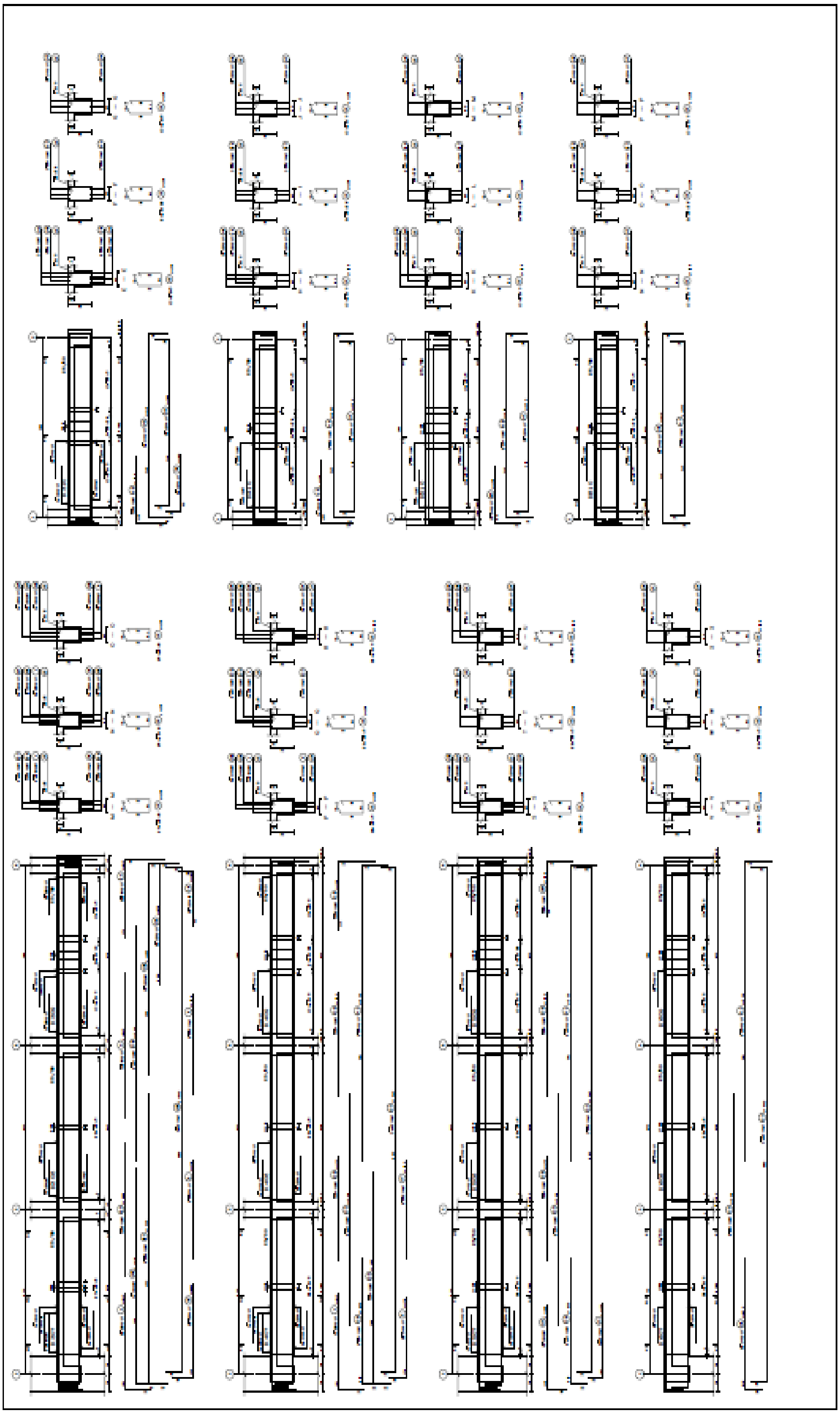

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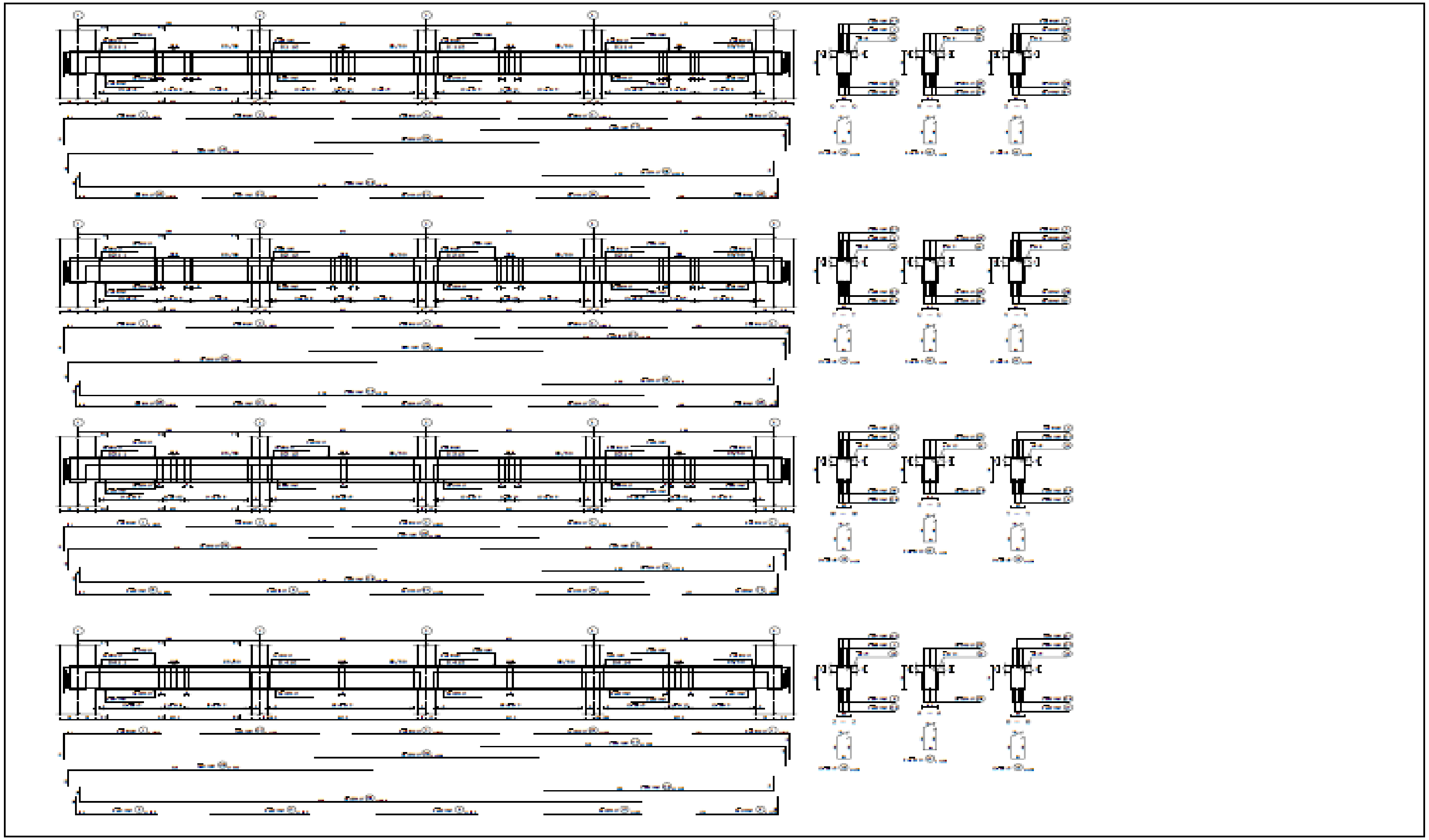

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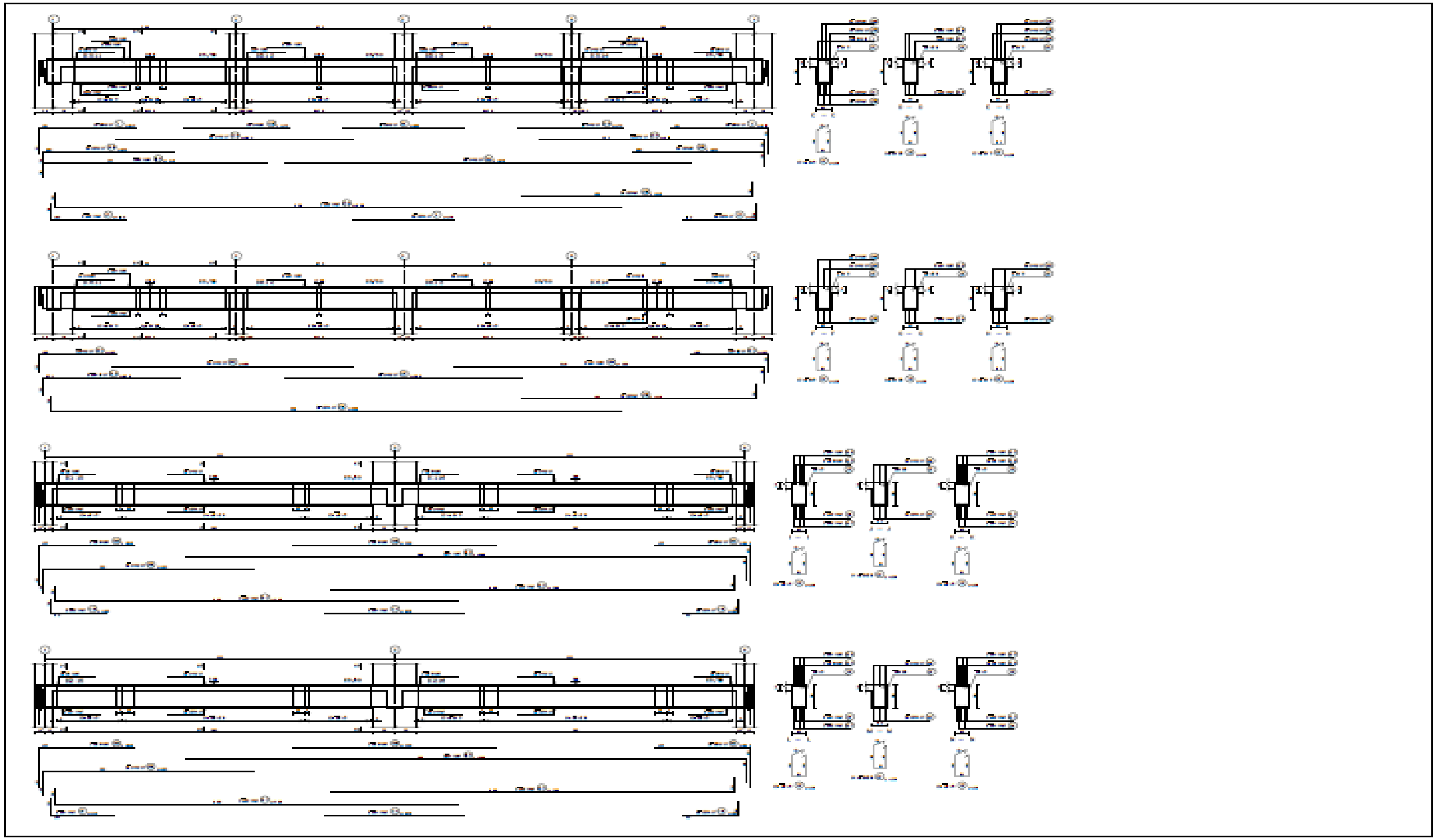

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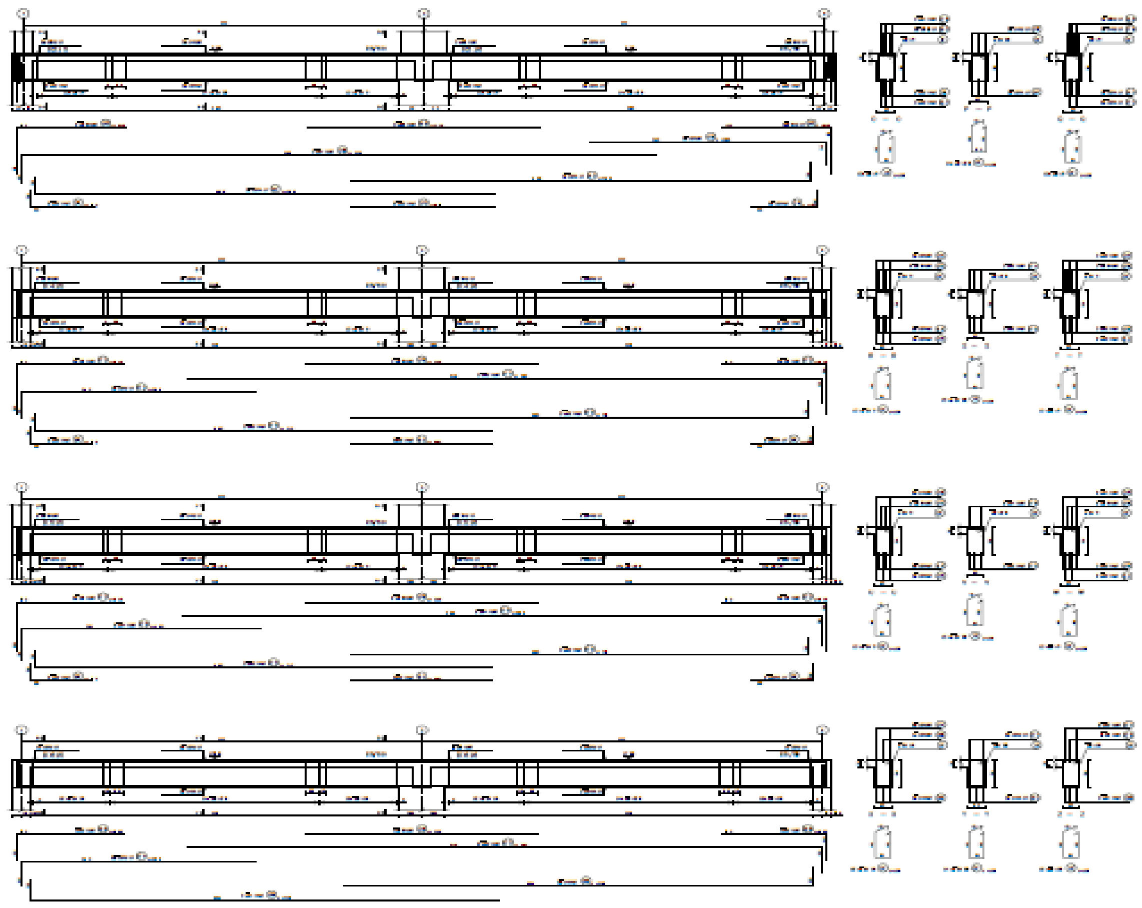

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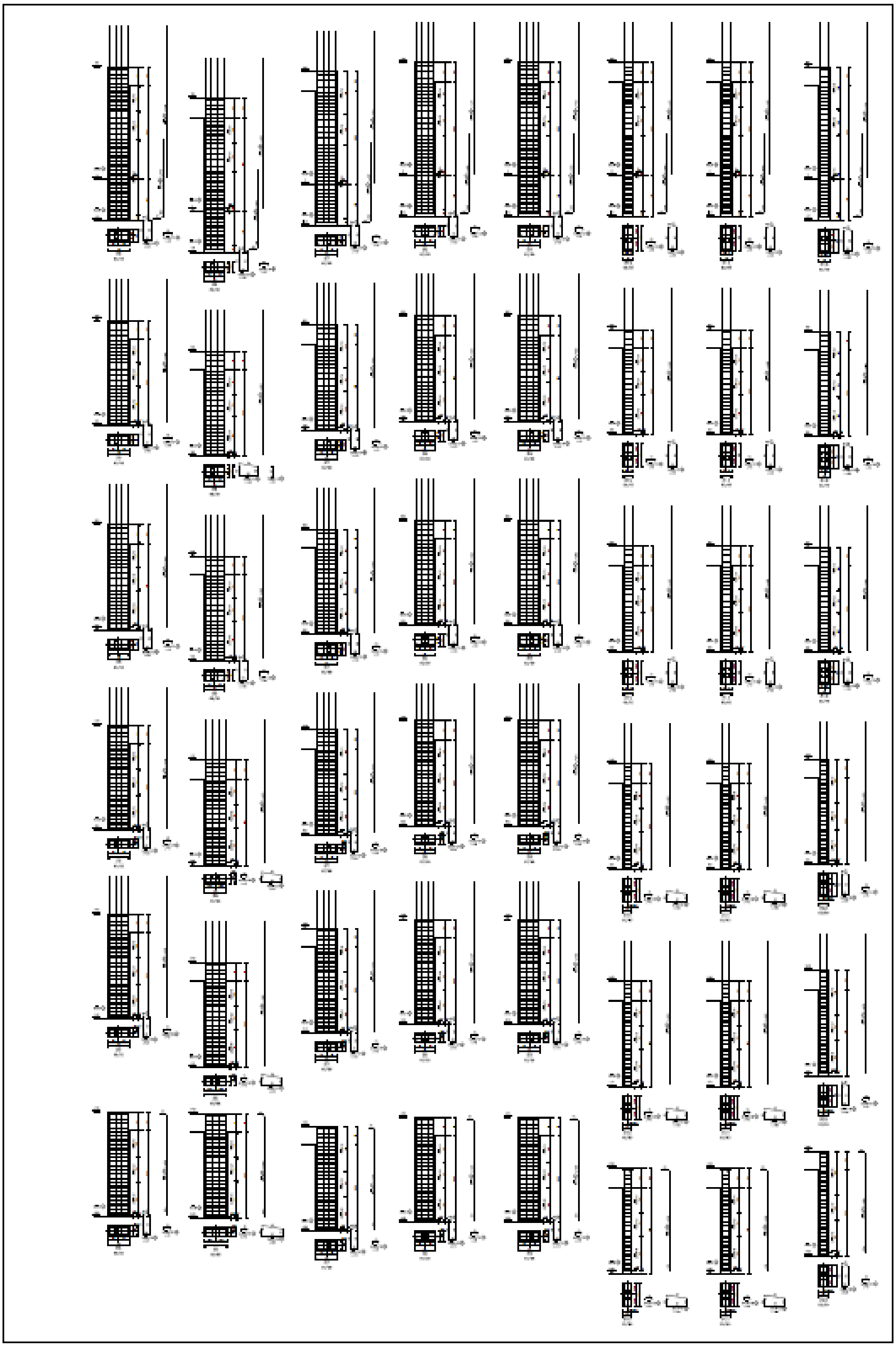
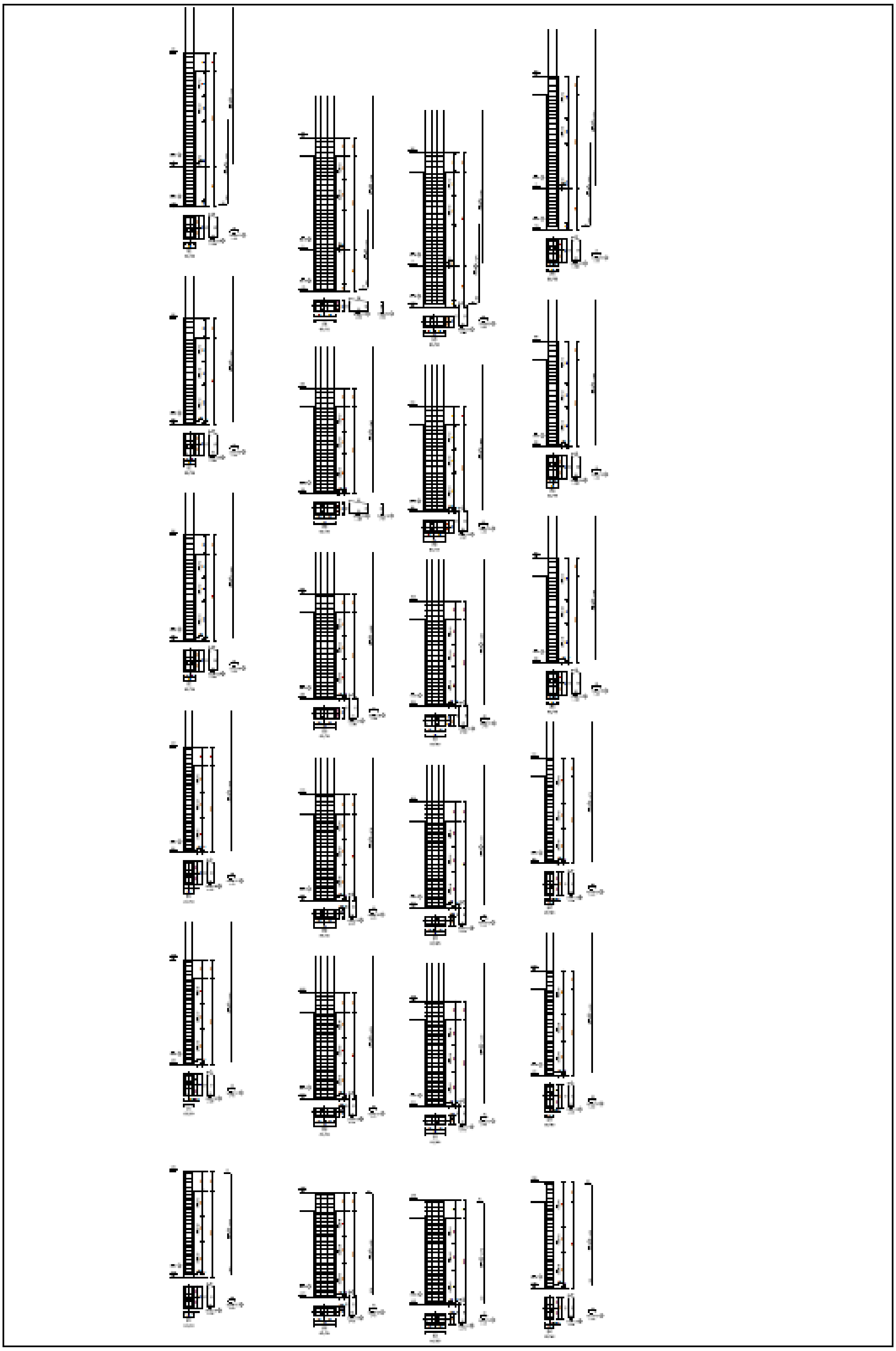


Figure A.39: Column Details of 6-Story Frame

Figure A.40: Column Details of 6-Story Frame



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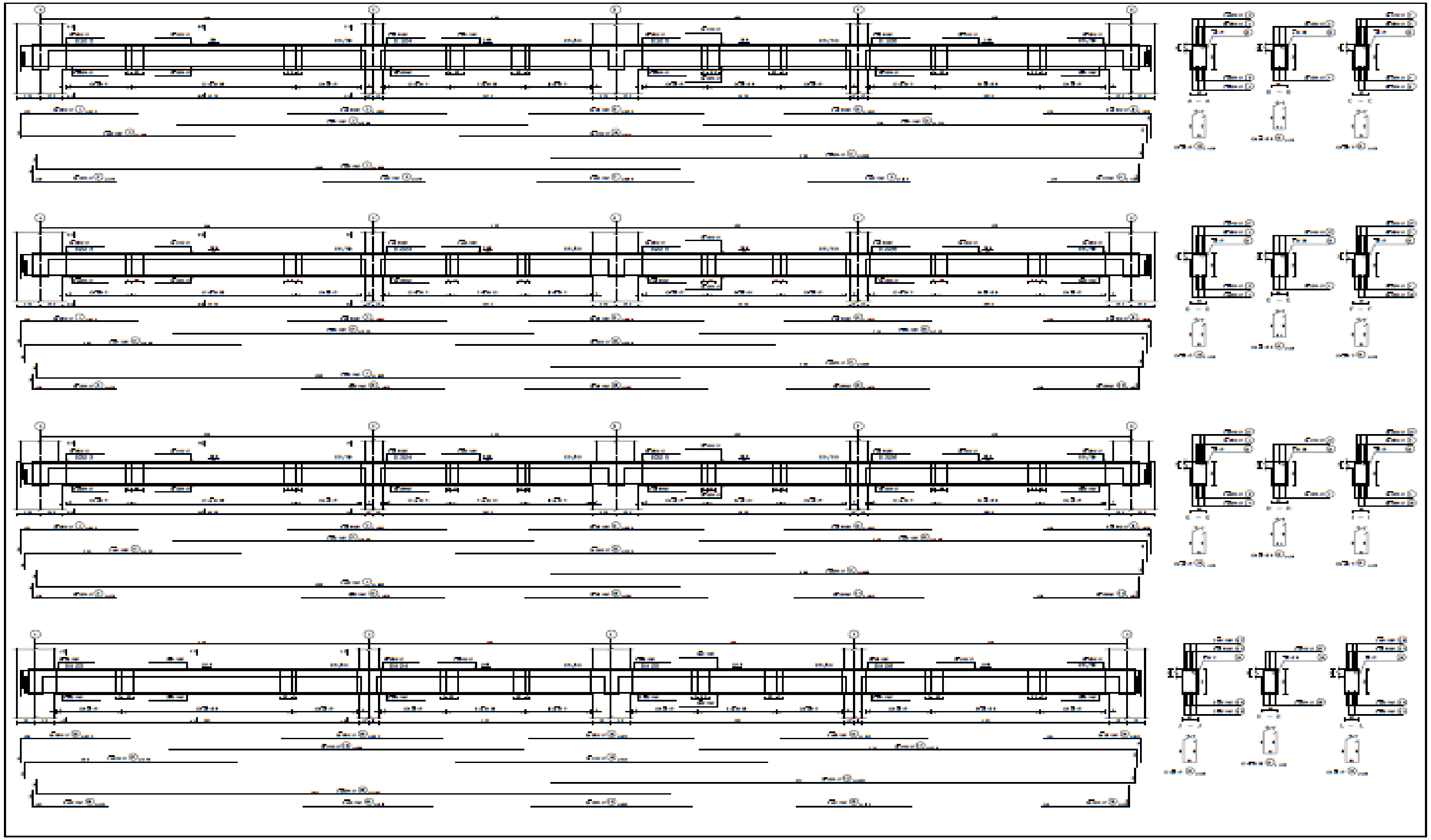
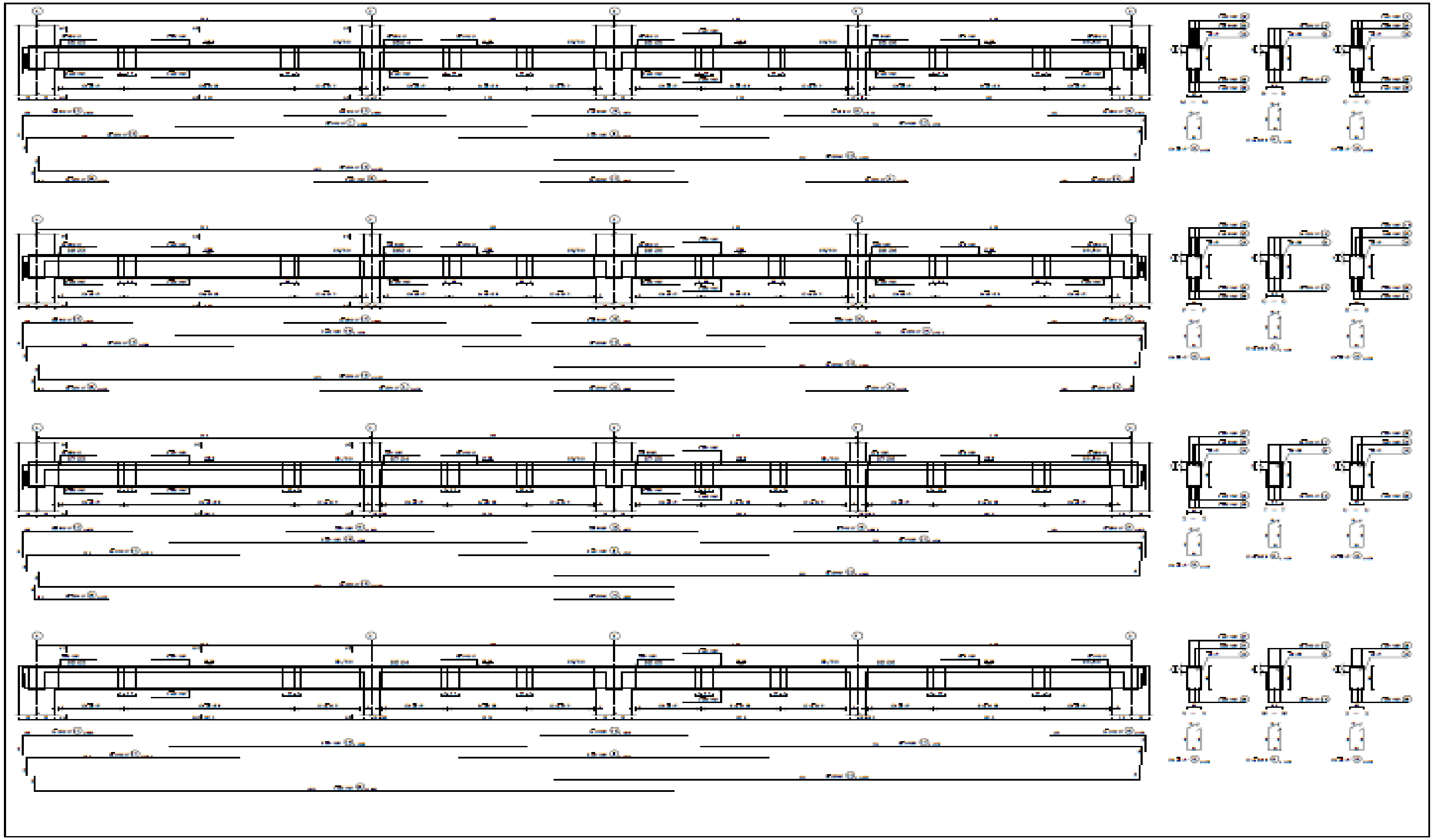


Figure A.42: Beam Details of 8-Story Frame



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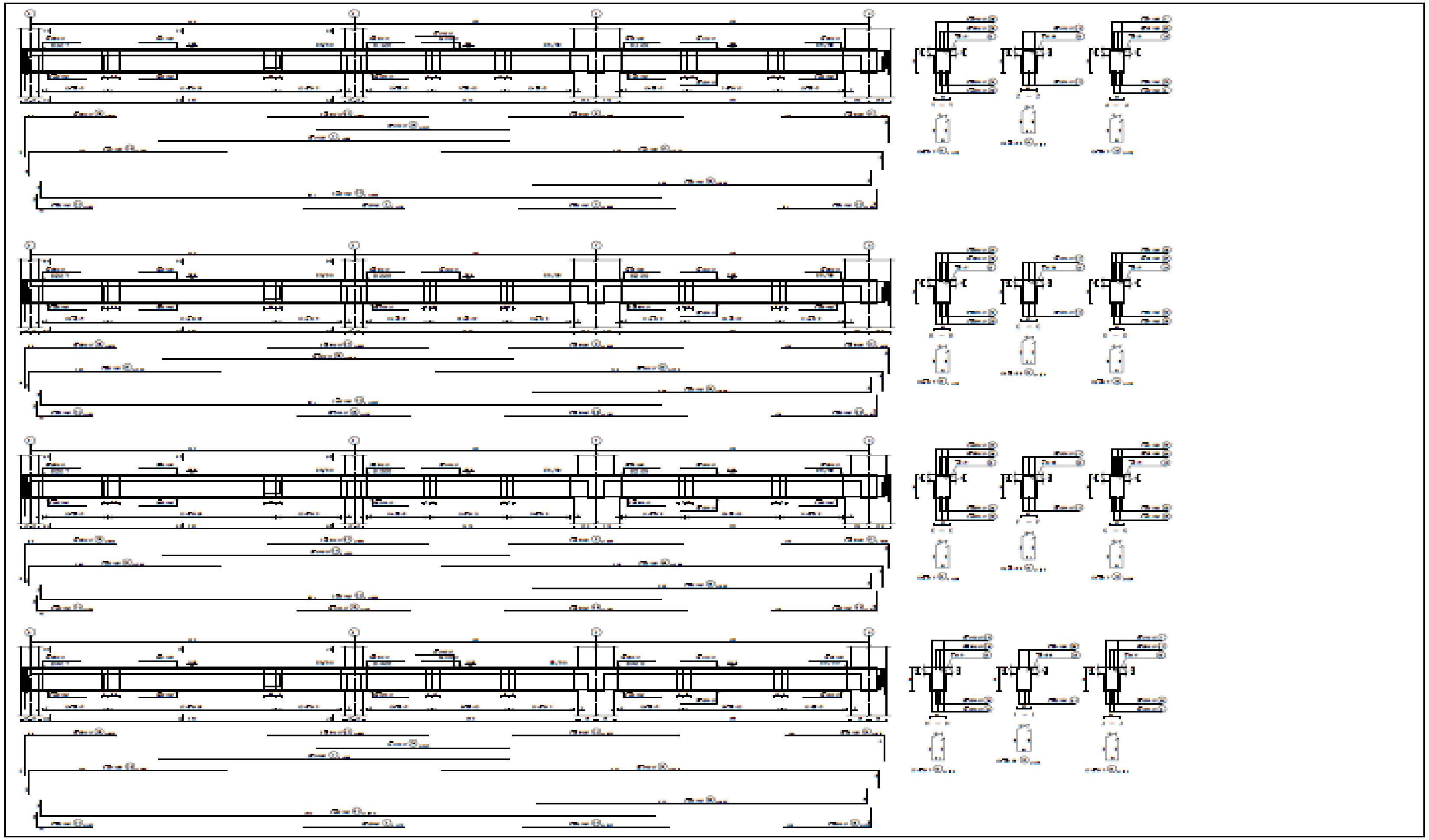

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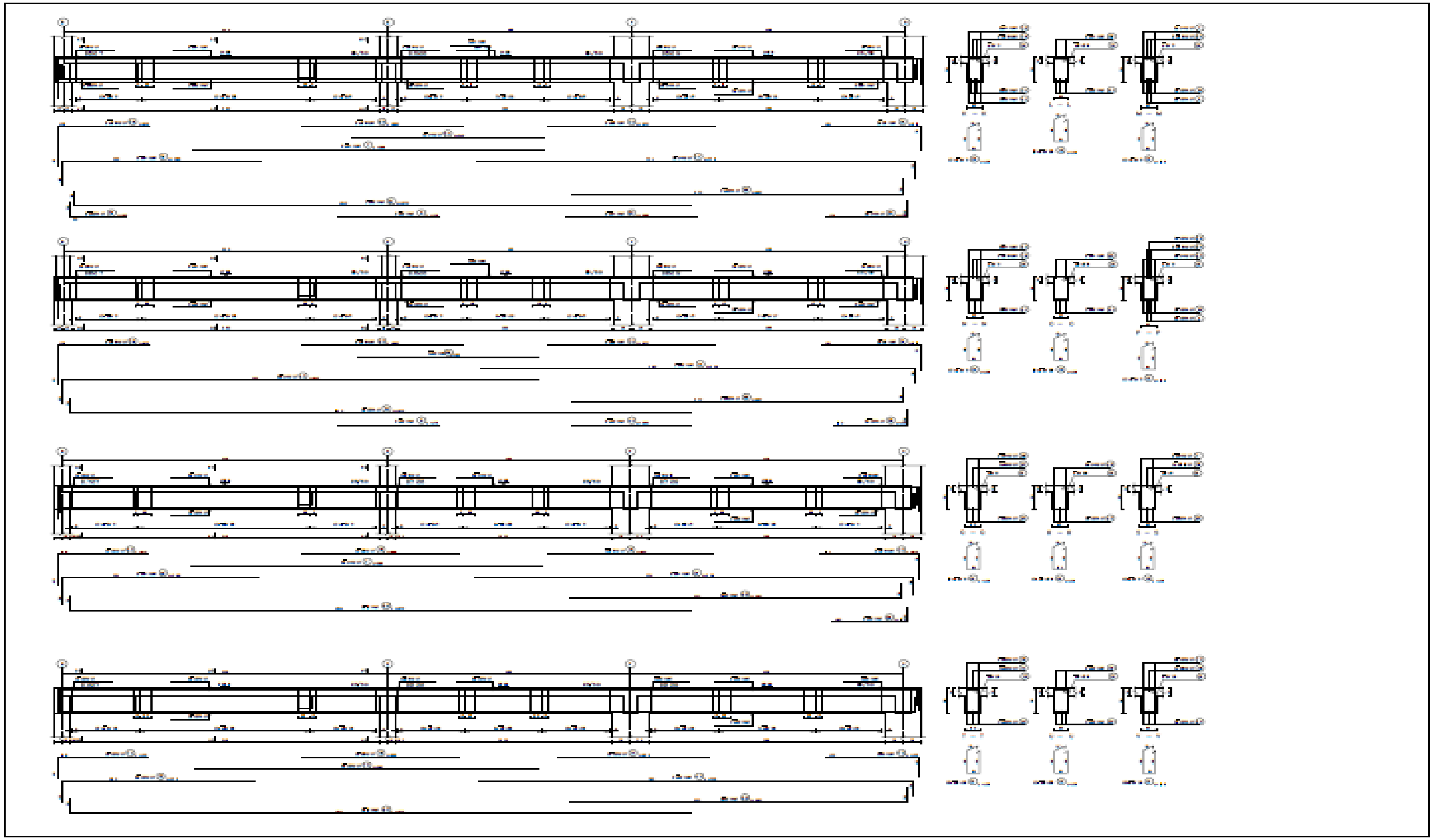

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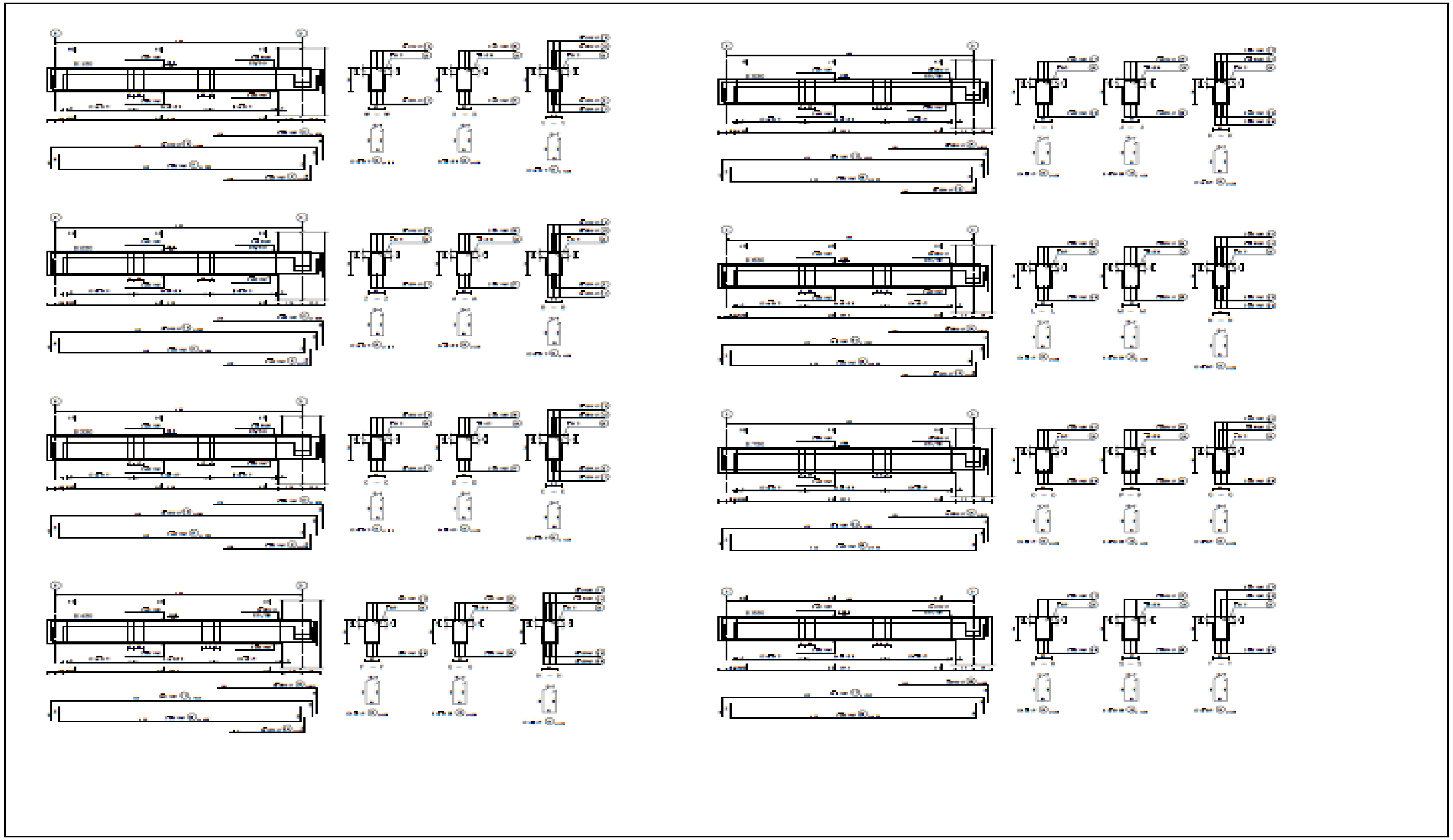

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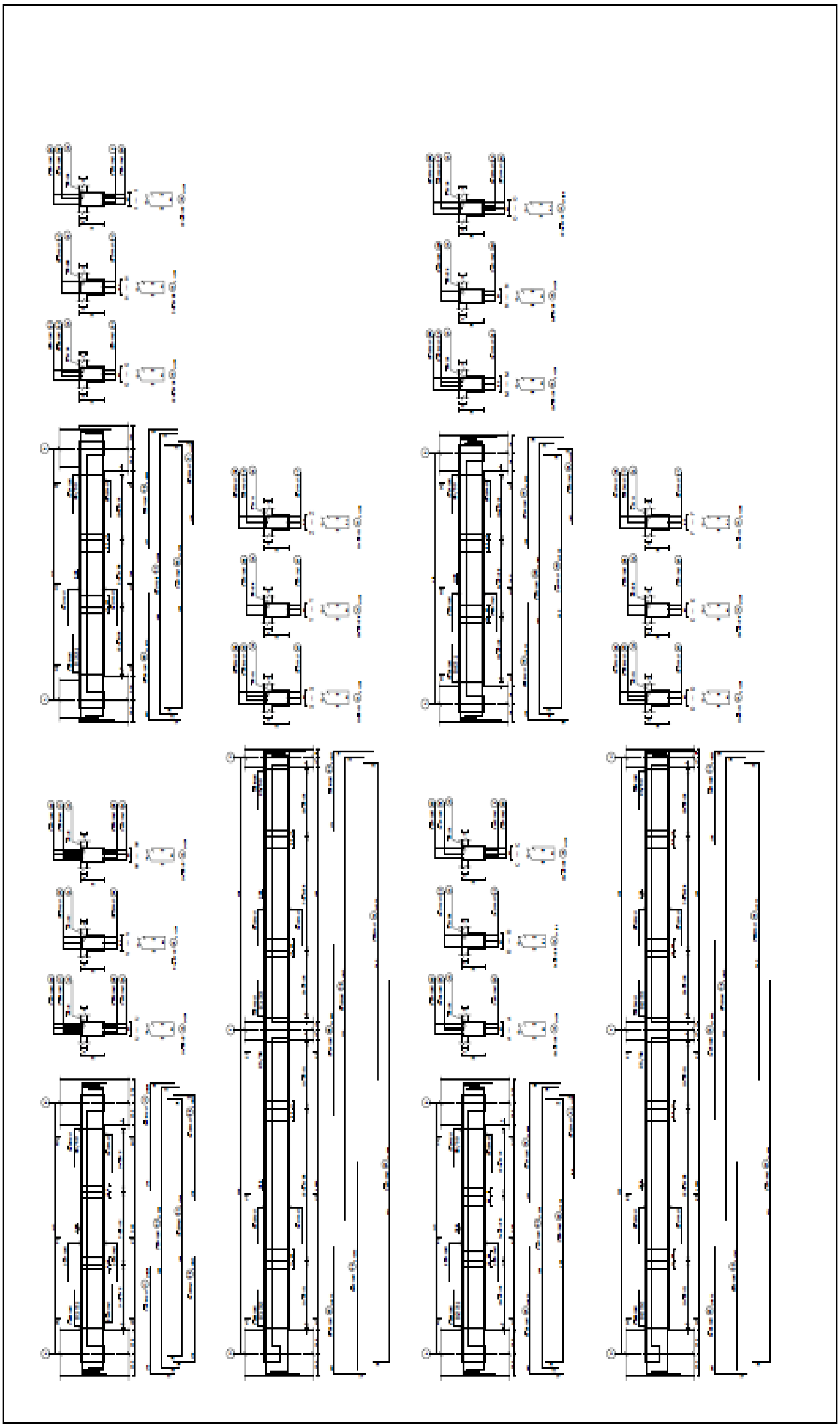

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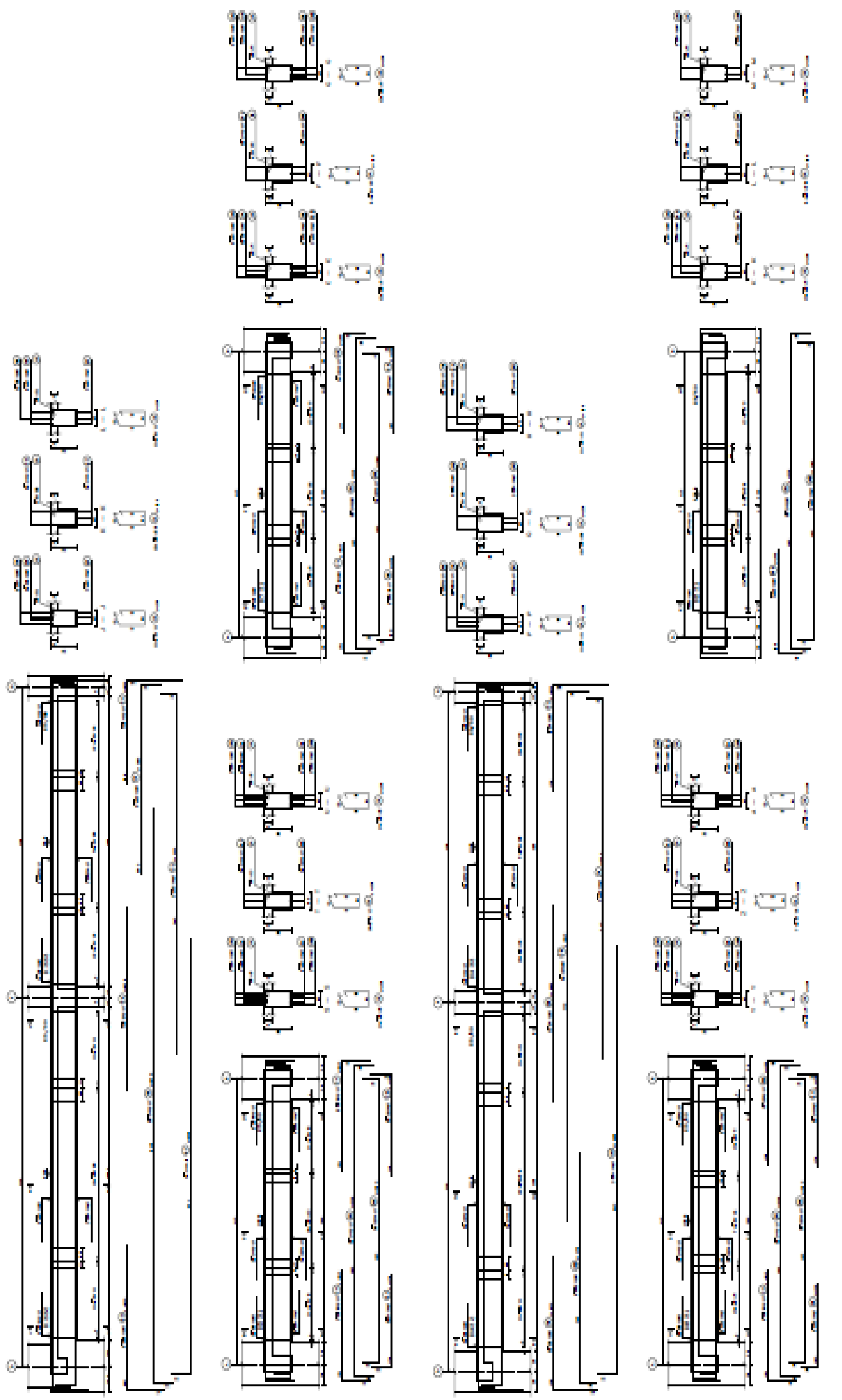

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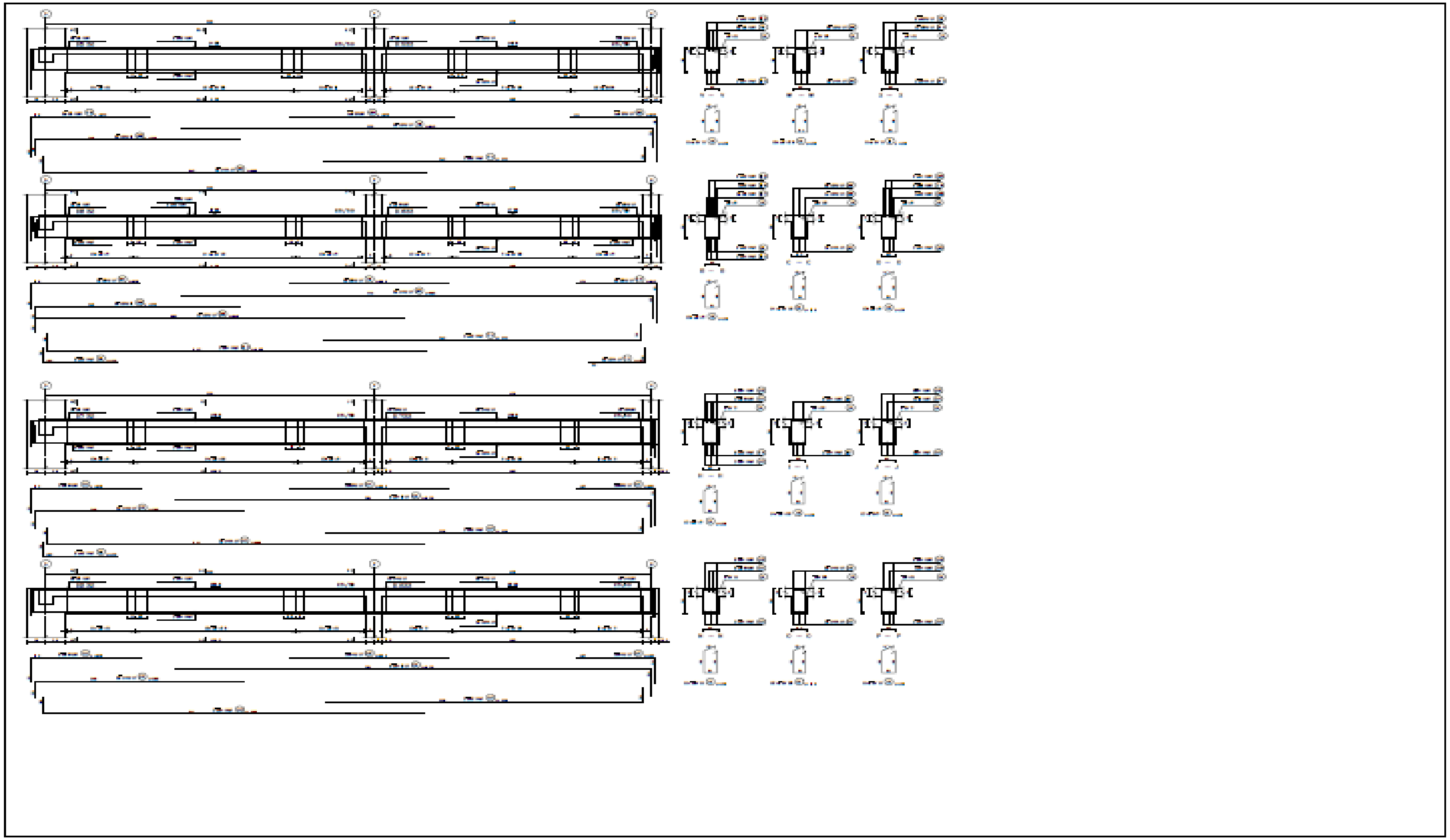

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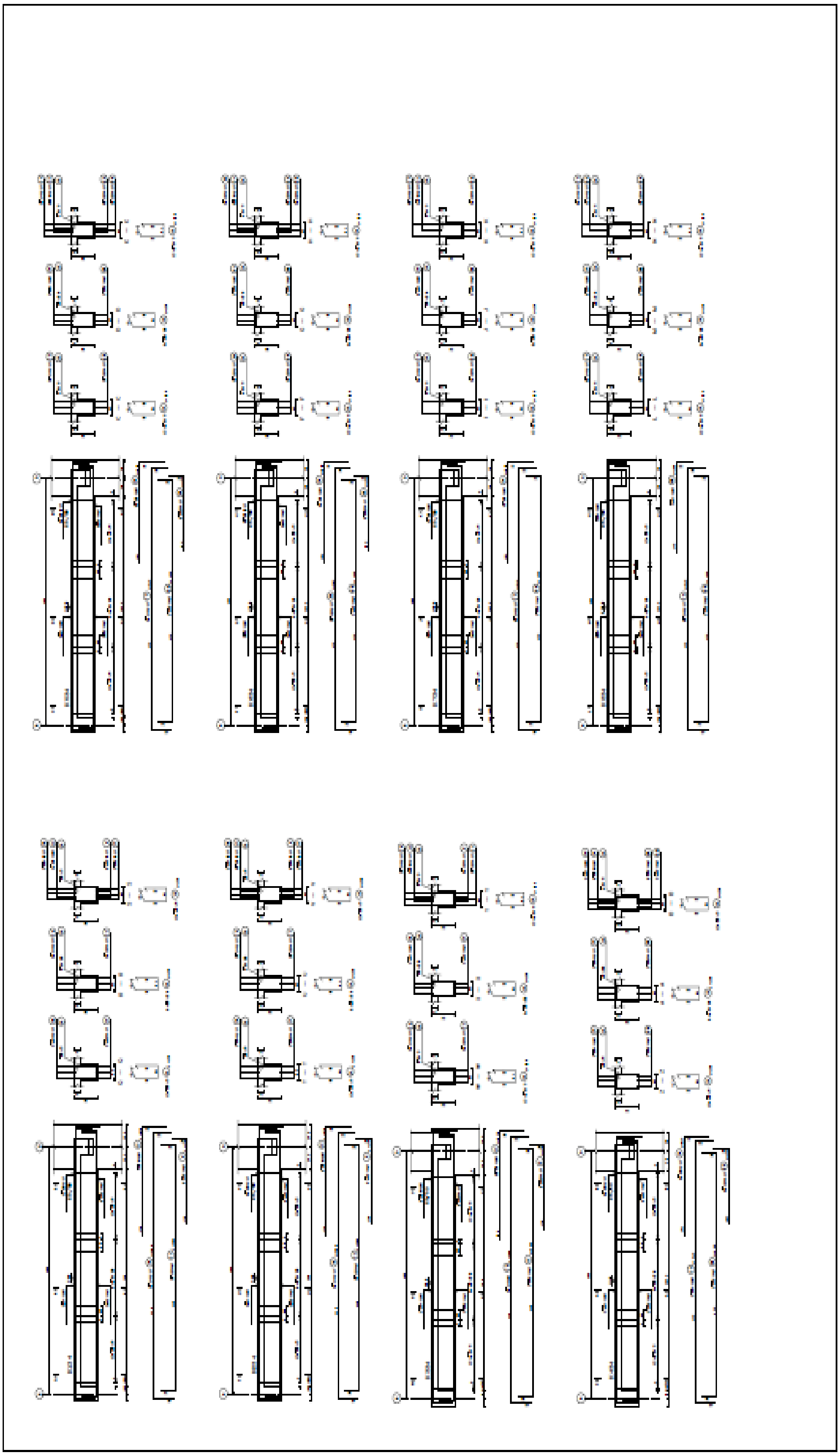

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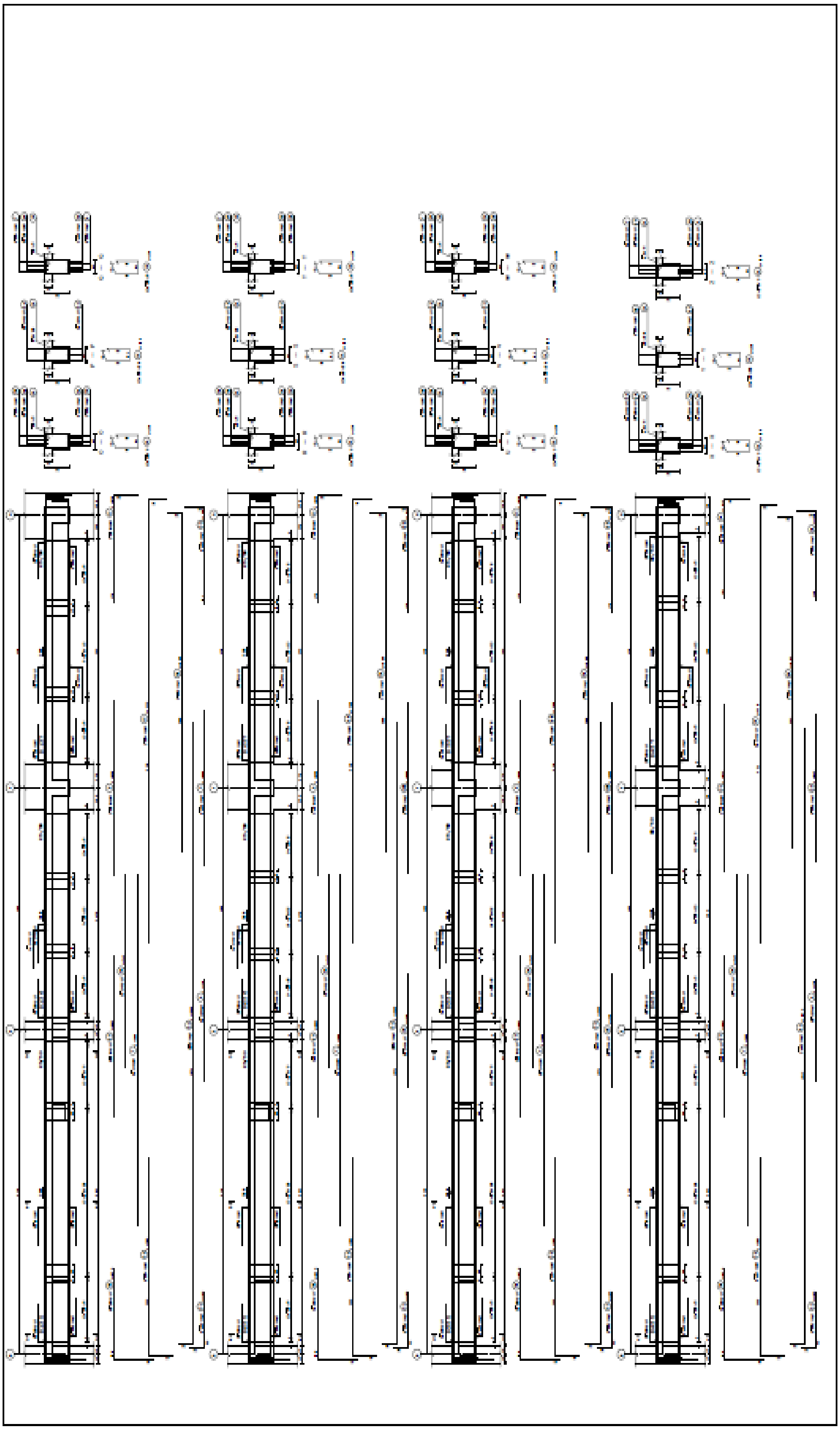

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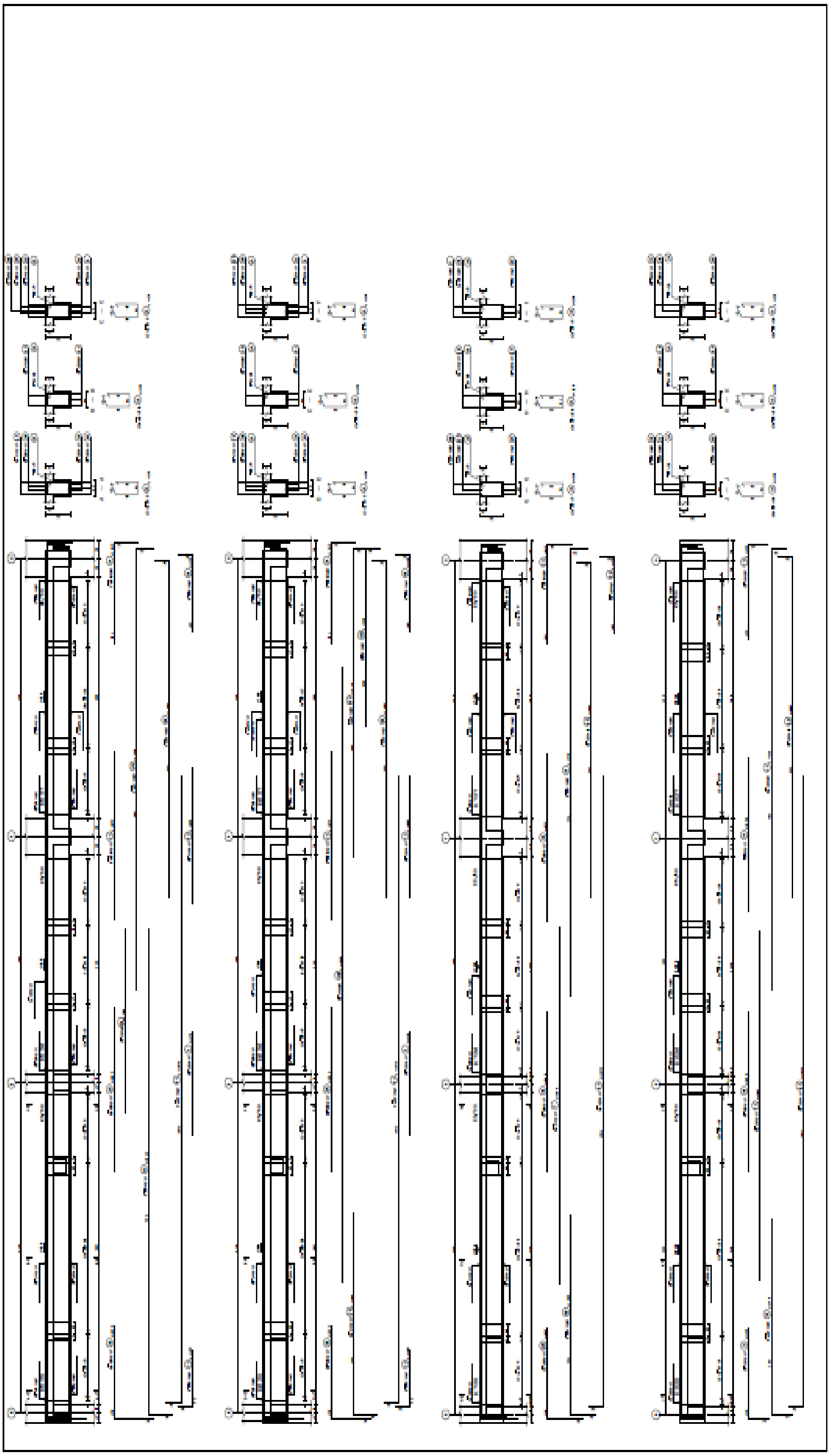

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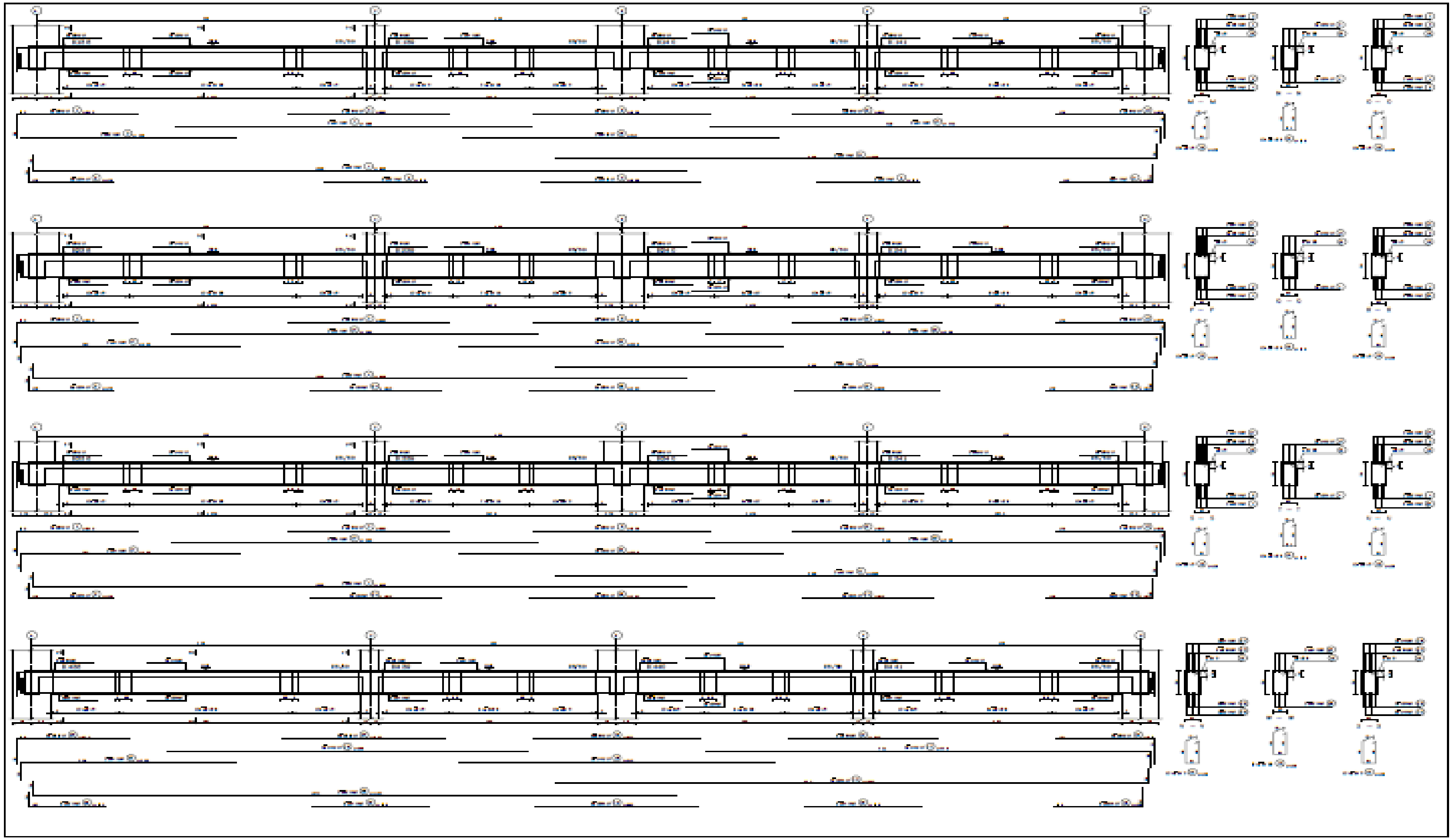

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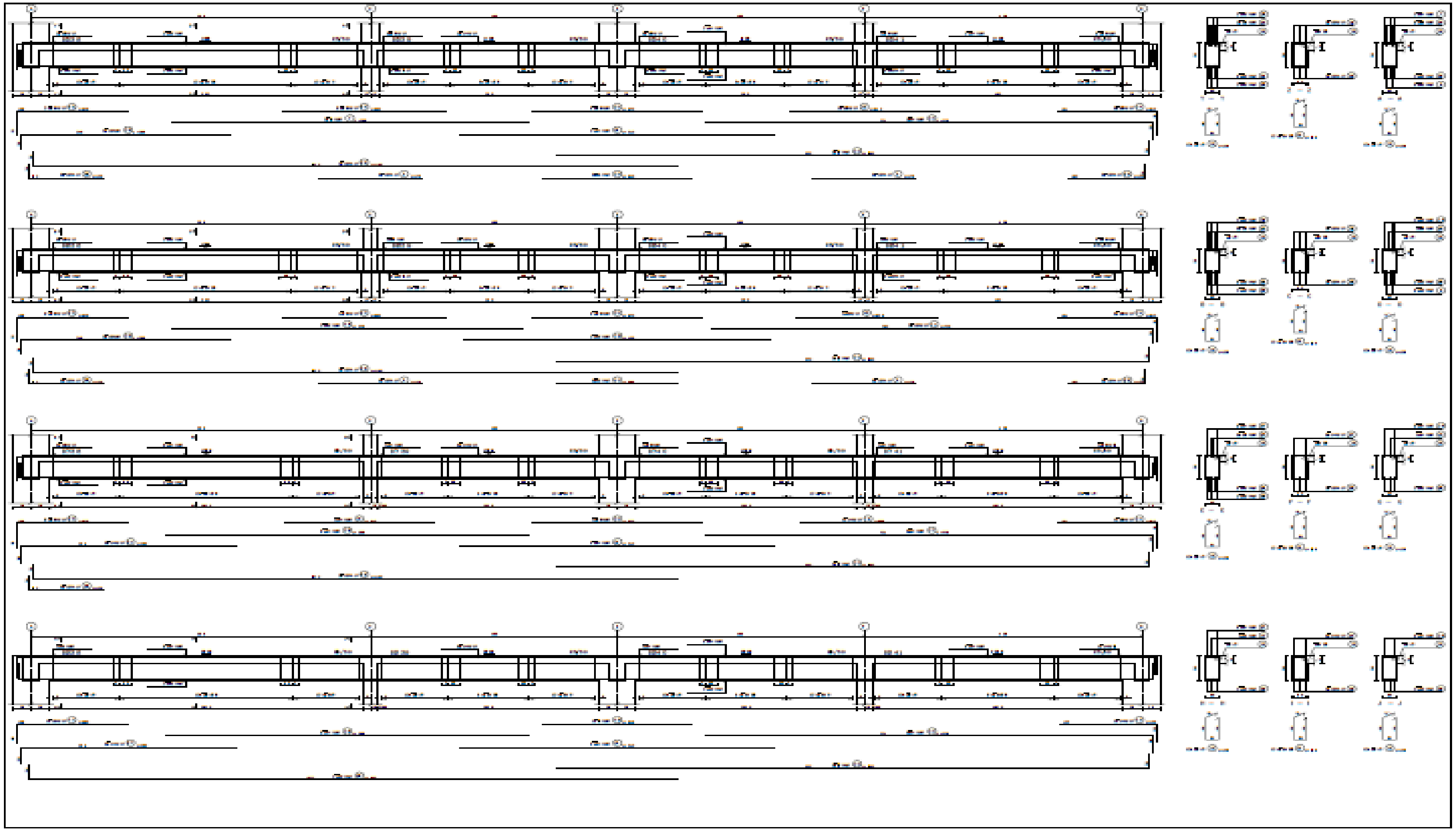

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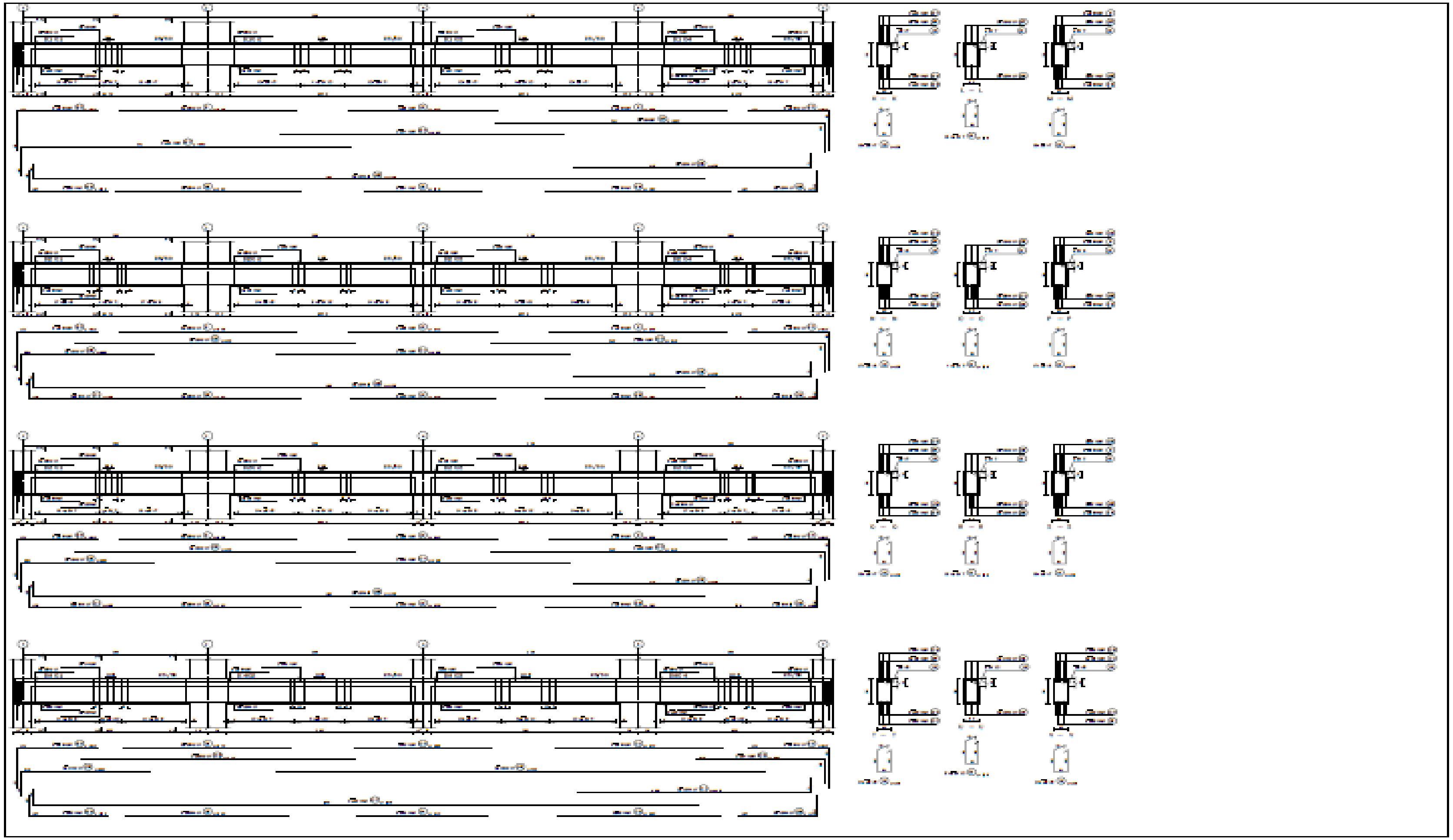

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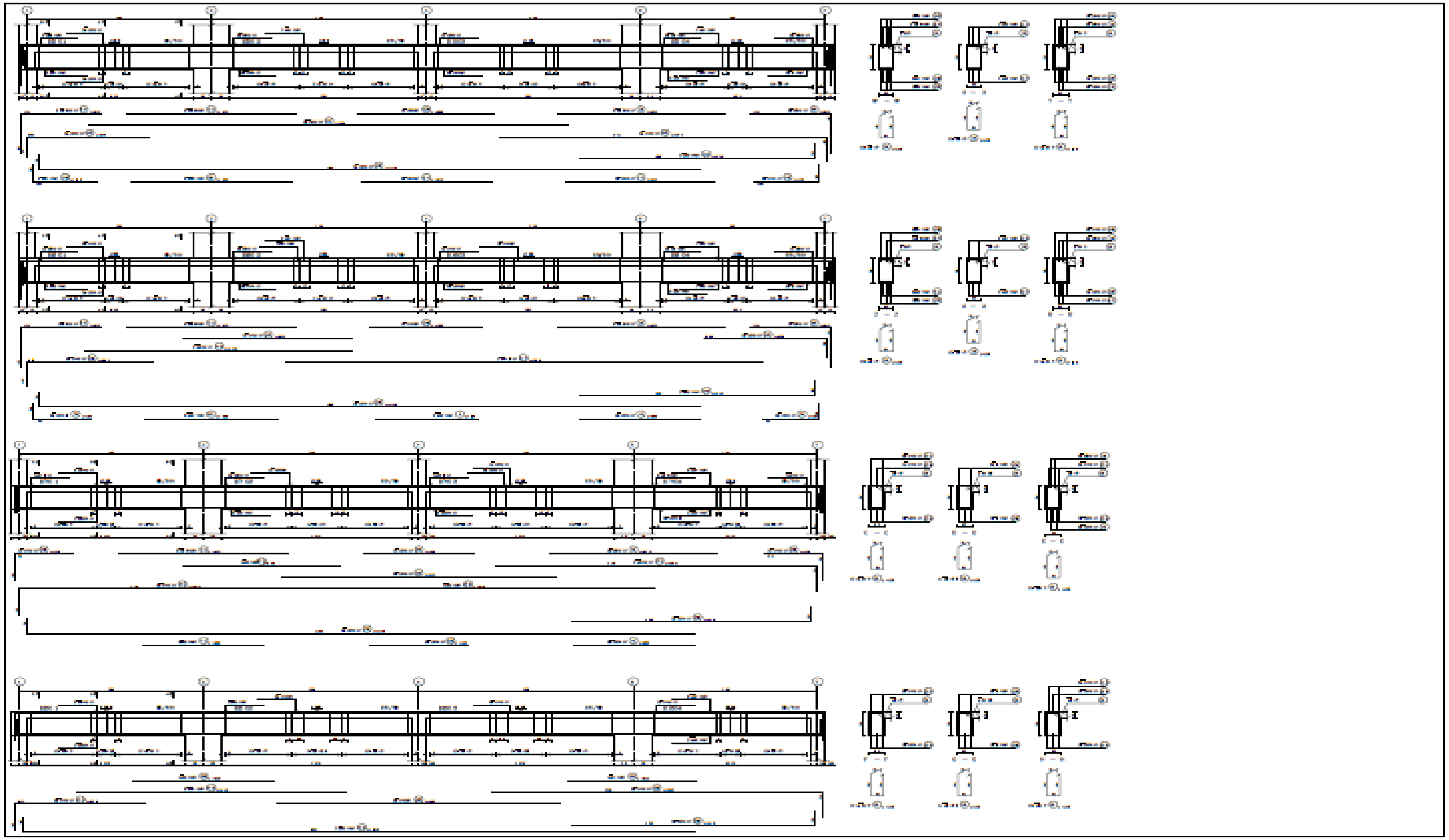

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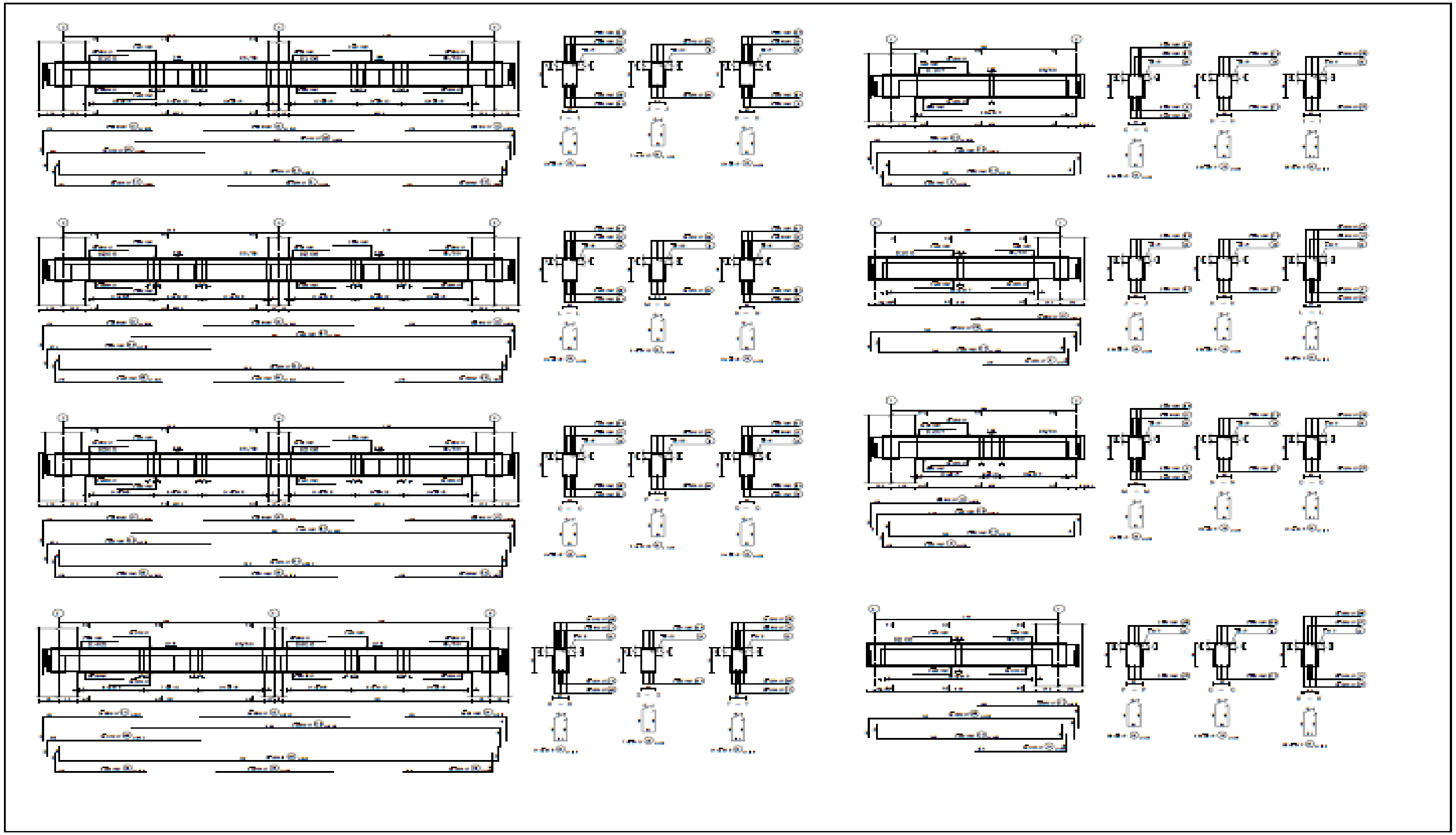

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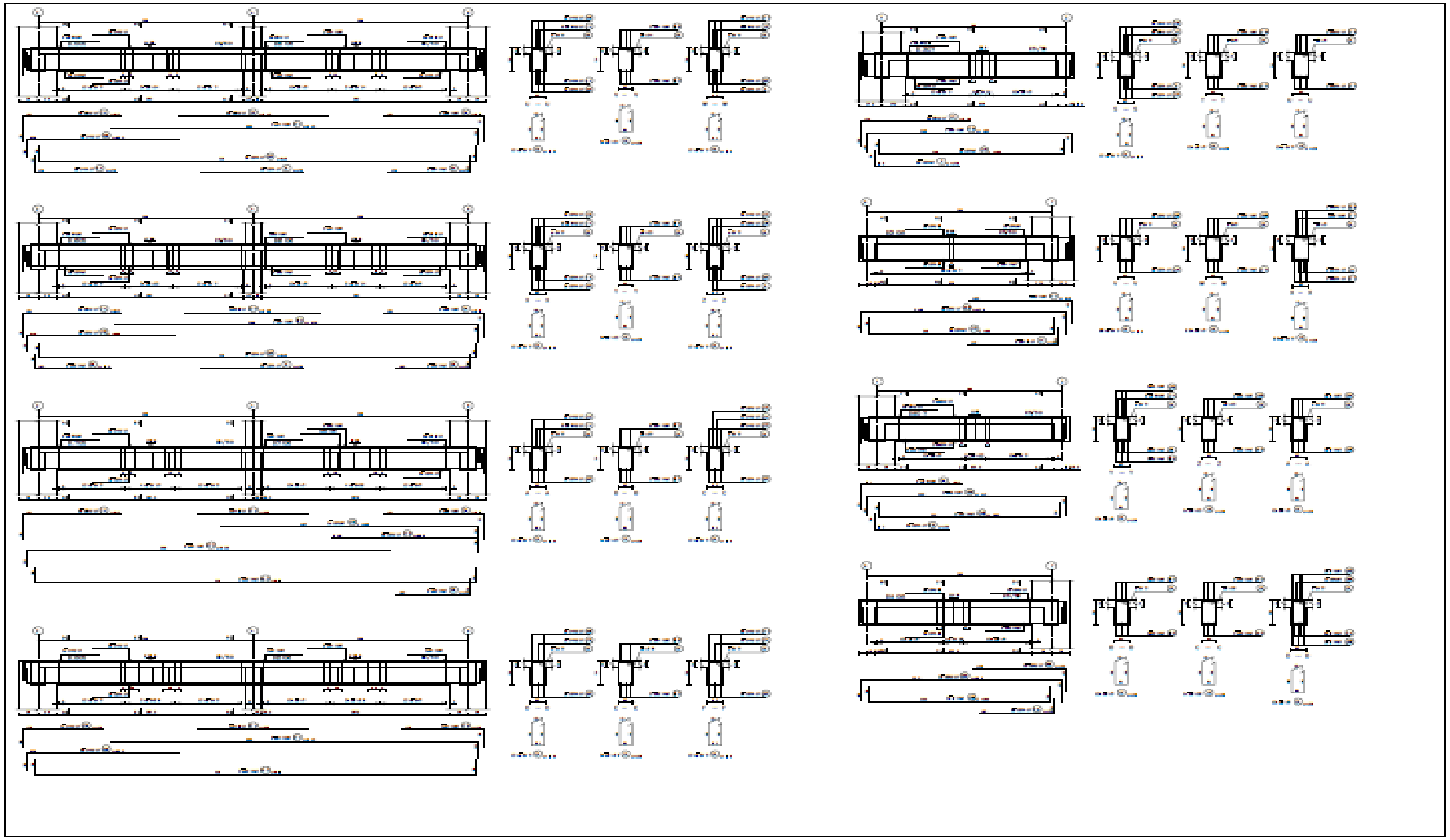

Figure A.58: Beam Details of 8-Story Frame

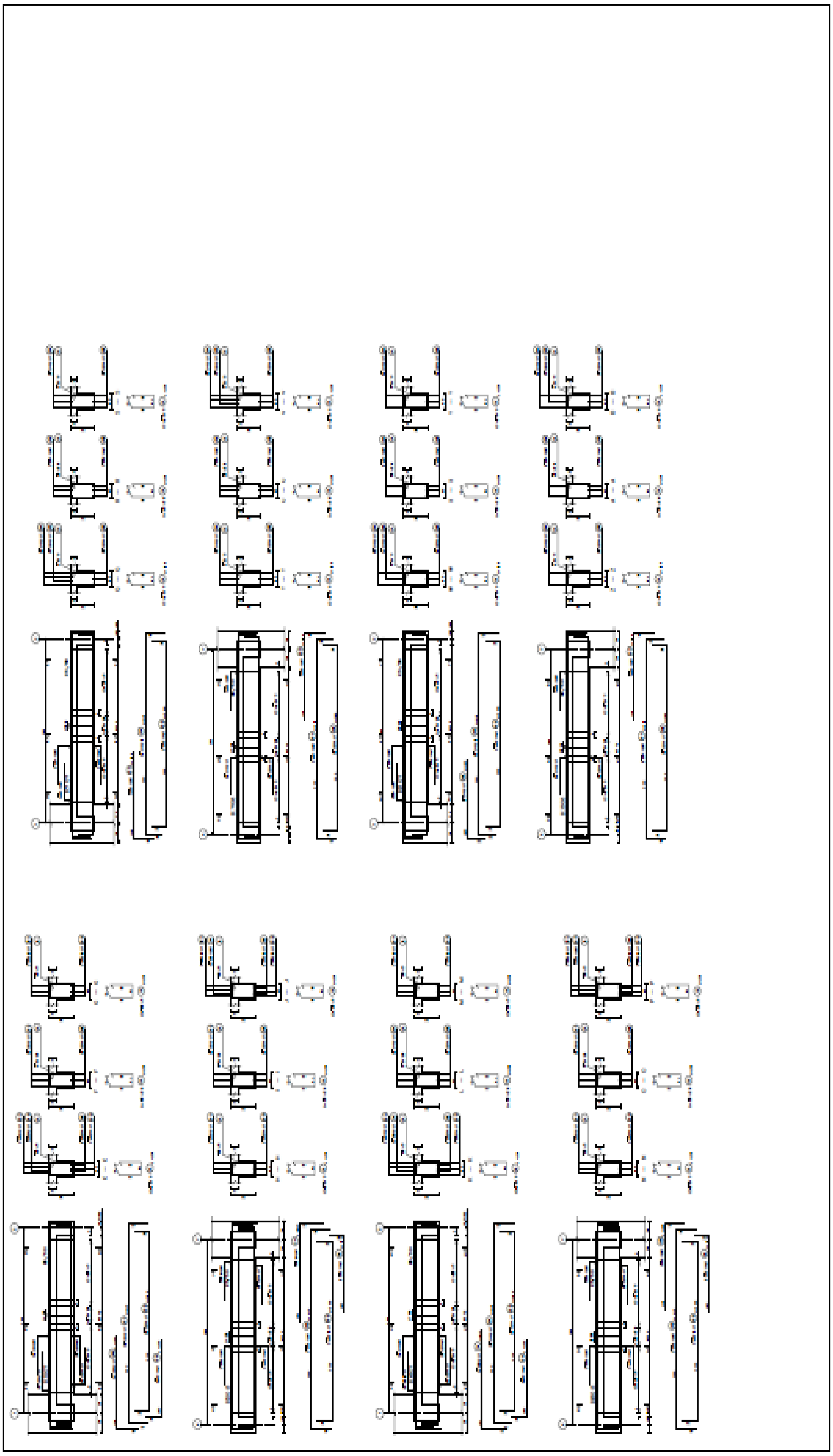

Figure A.59: Beam Details of 8-Story Frame

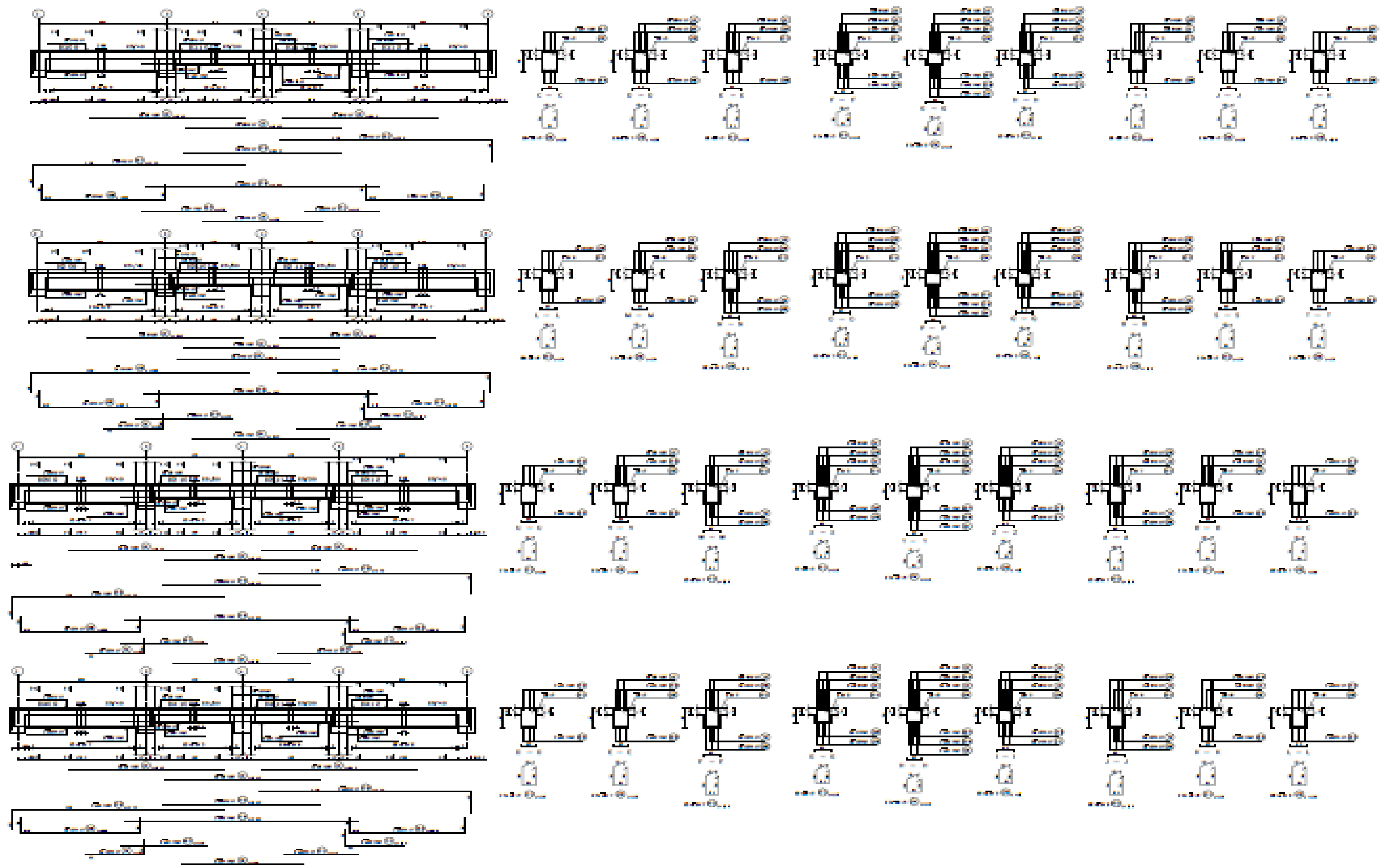

Figure A.60: Beam Details of 8-Story Frame

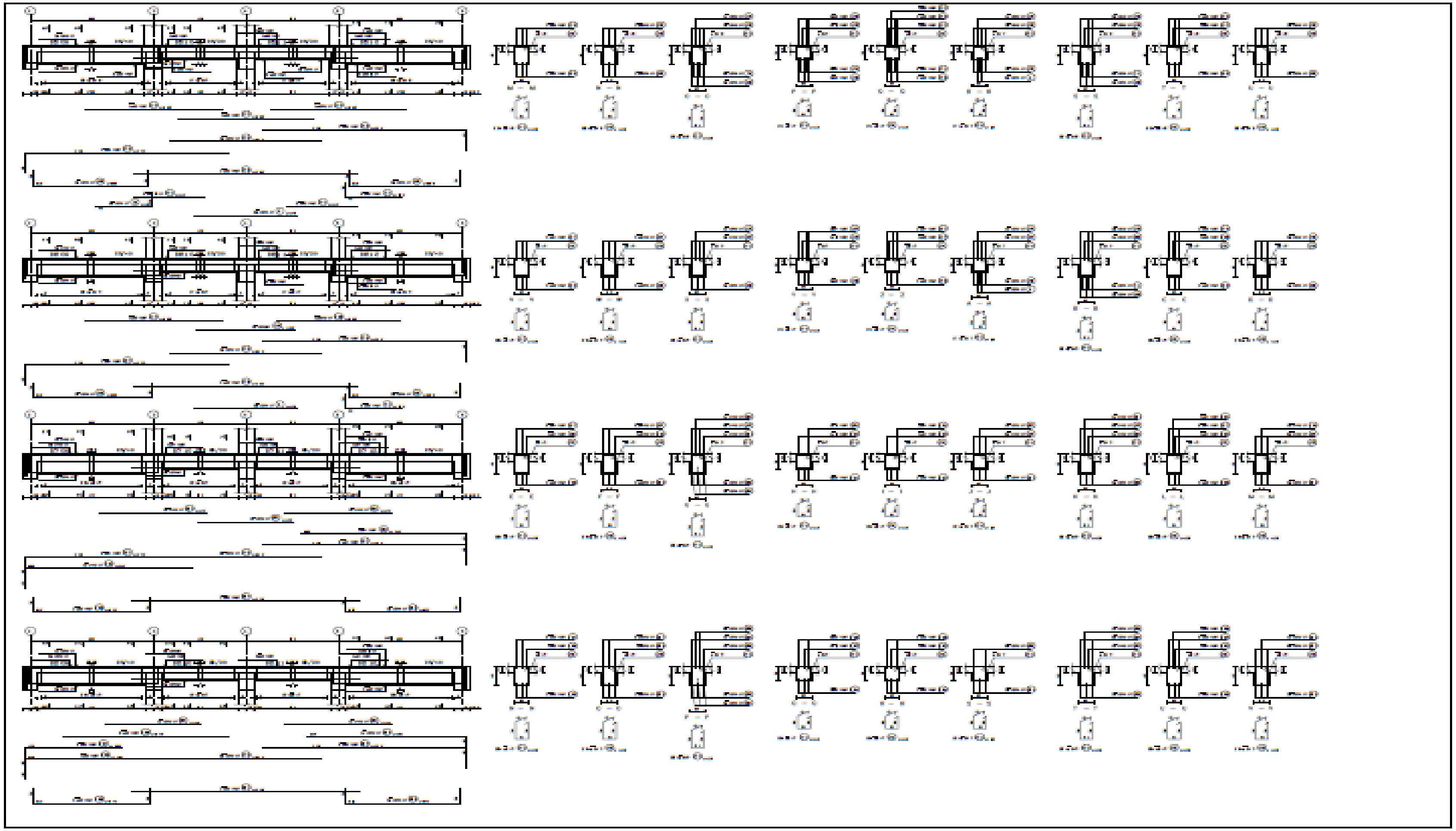

Figure A.61: Beam Details of 8-Story Frame

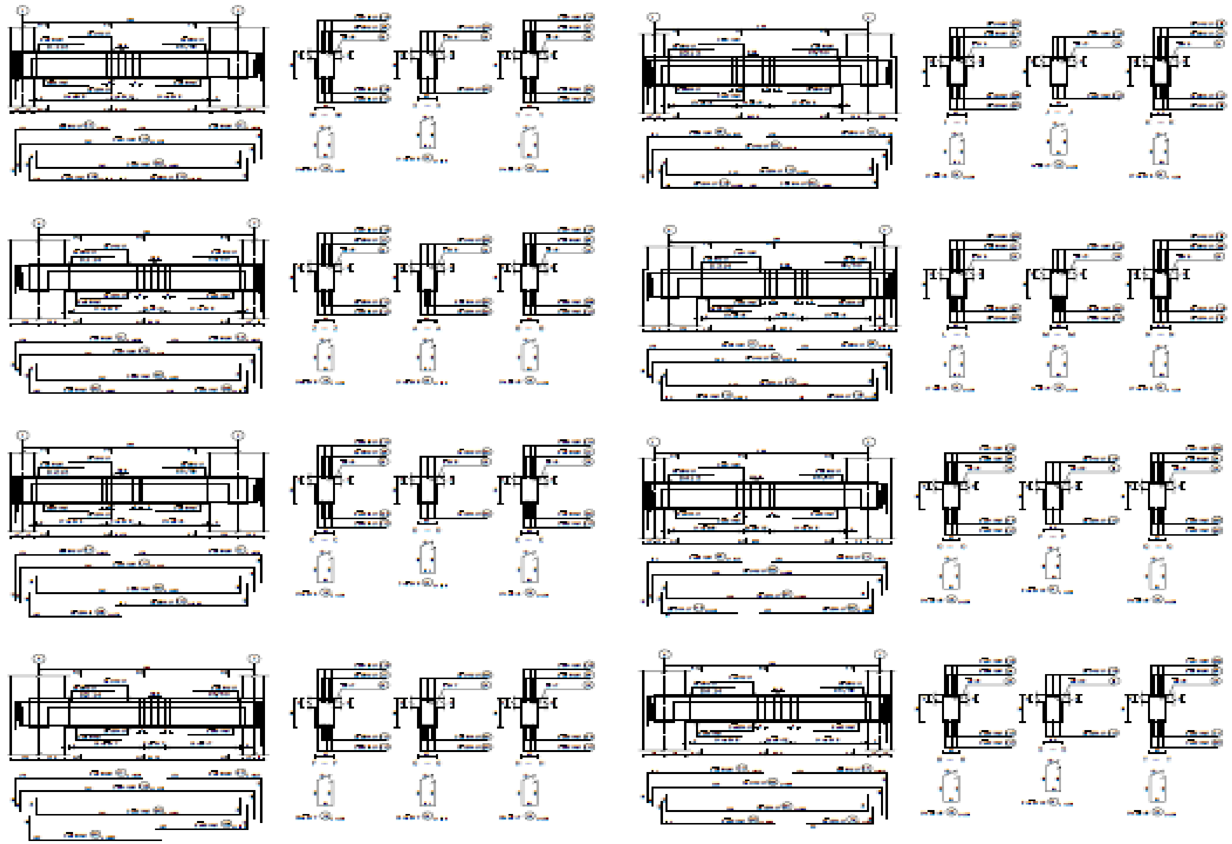

Figure A.62: Beam Details of 8-Story Frame

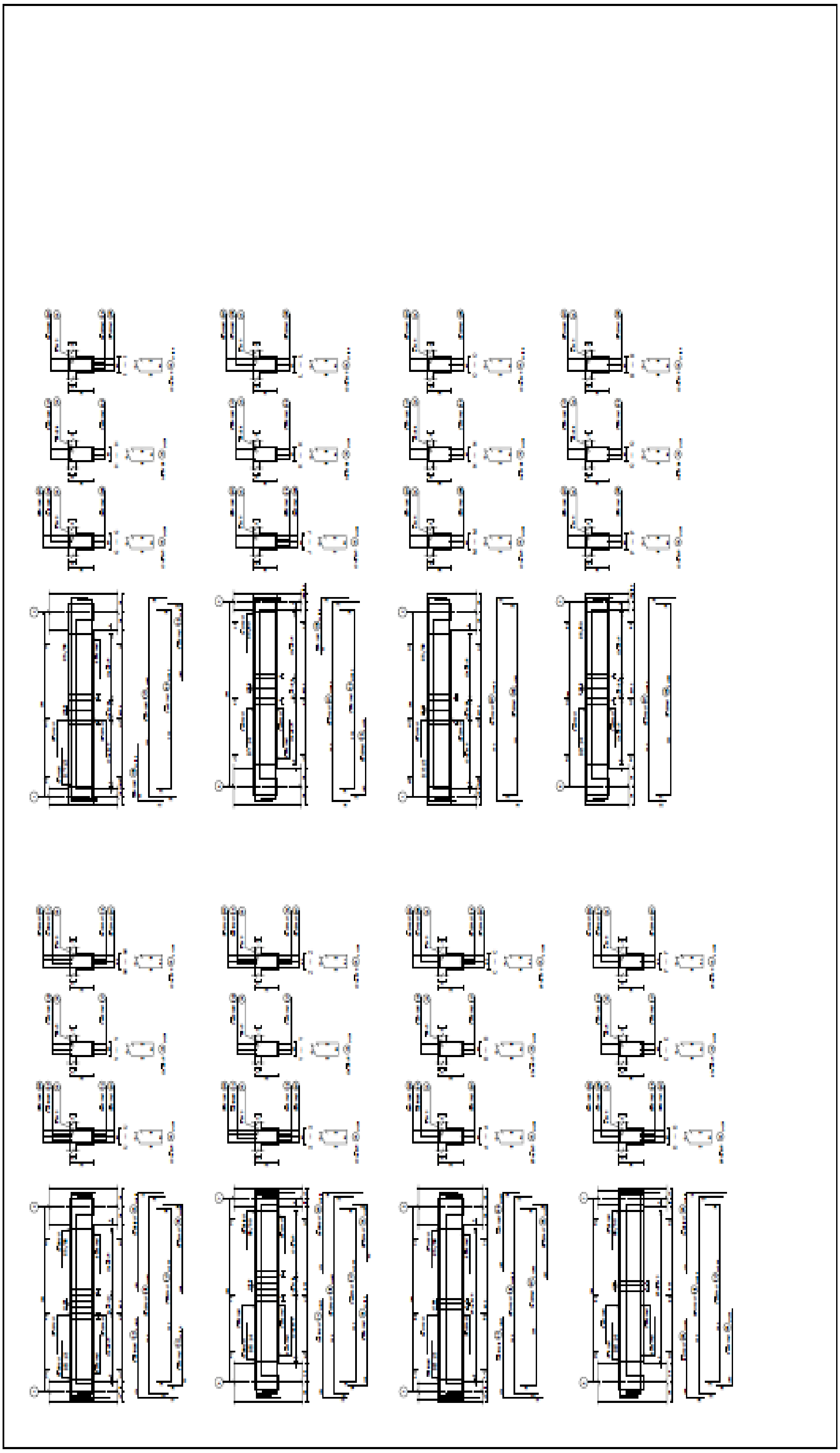

Figure A.63: Beam Details of 8-Story Frame

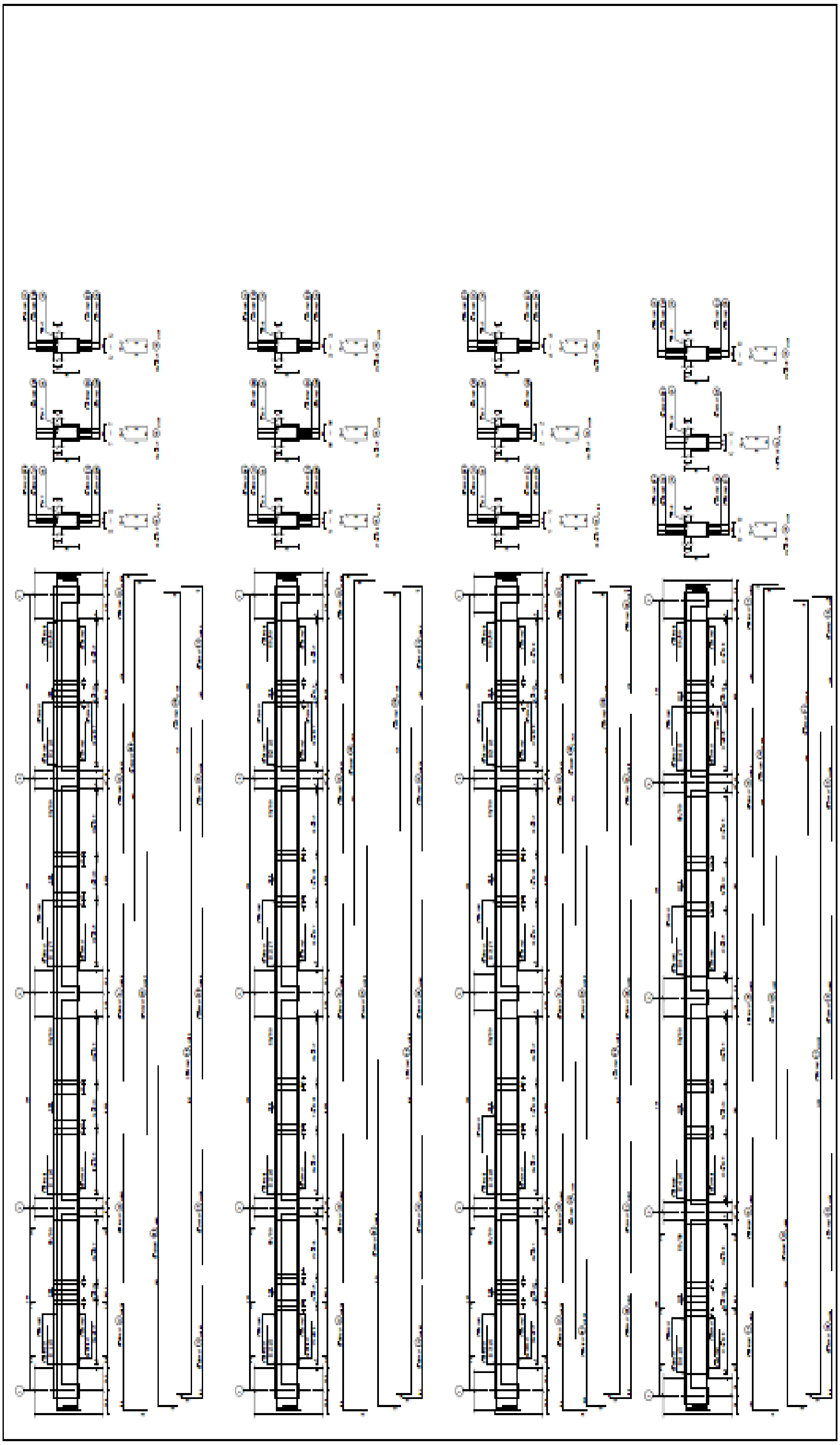

Figure A.64: Beam Details of 8-Story Frame

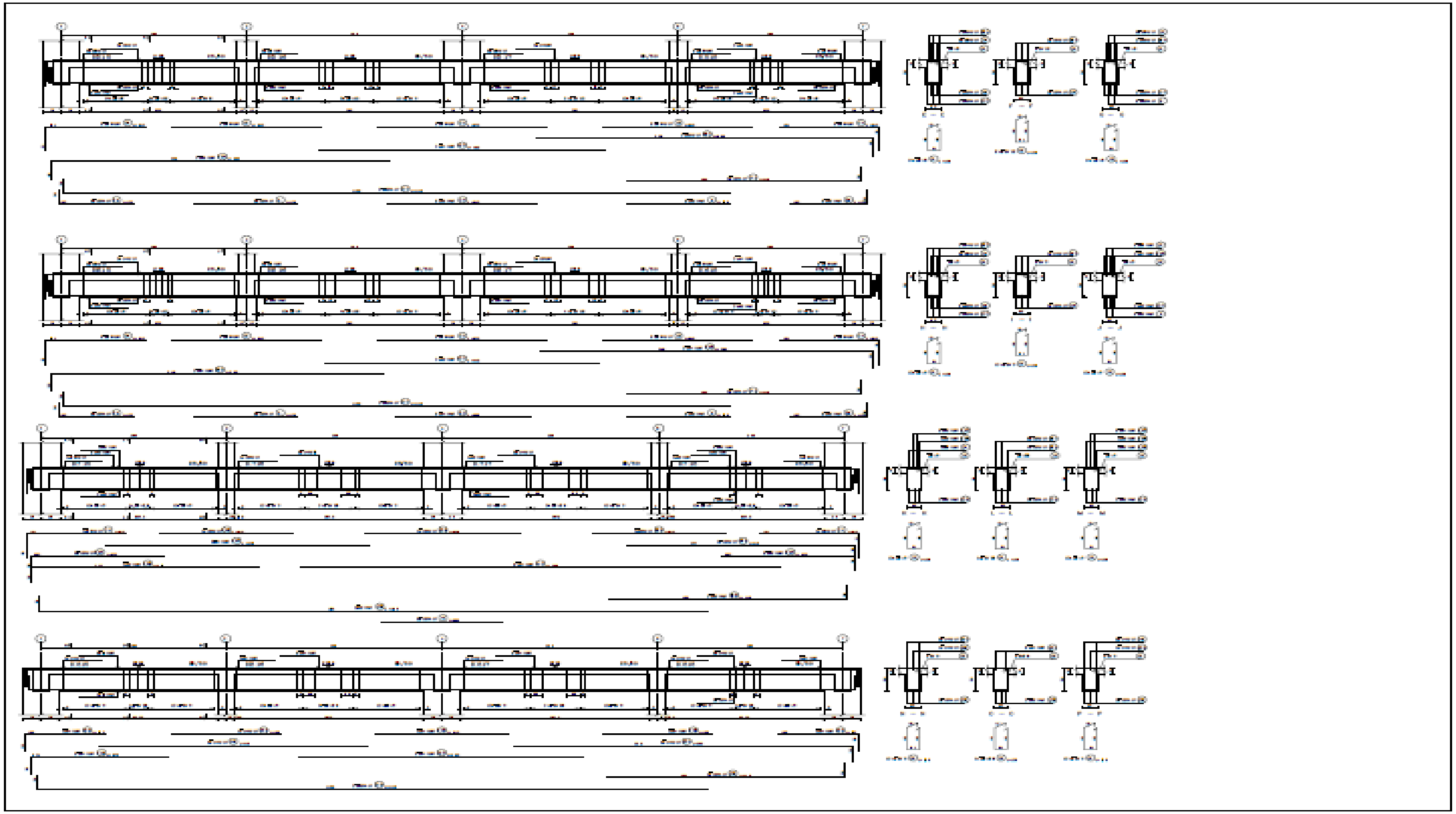

Figure A.65: Beam Details of 8-Story Frame

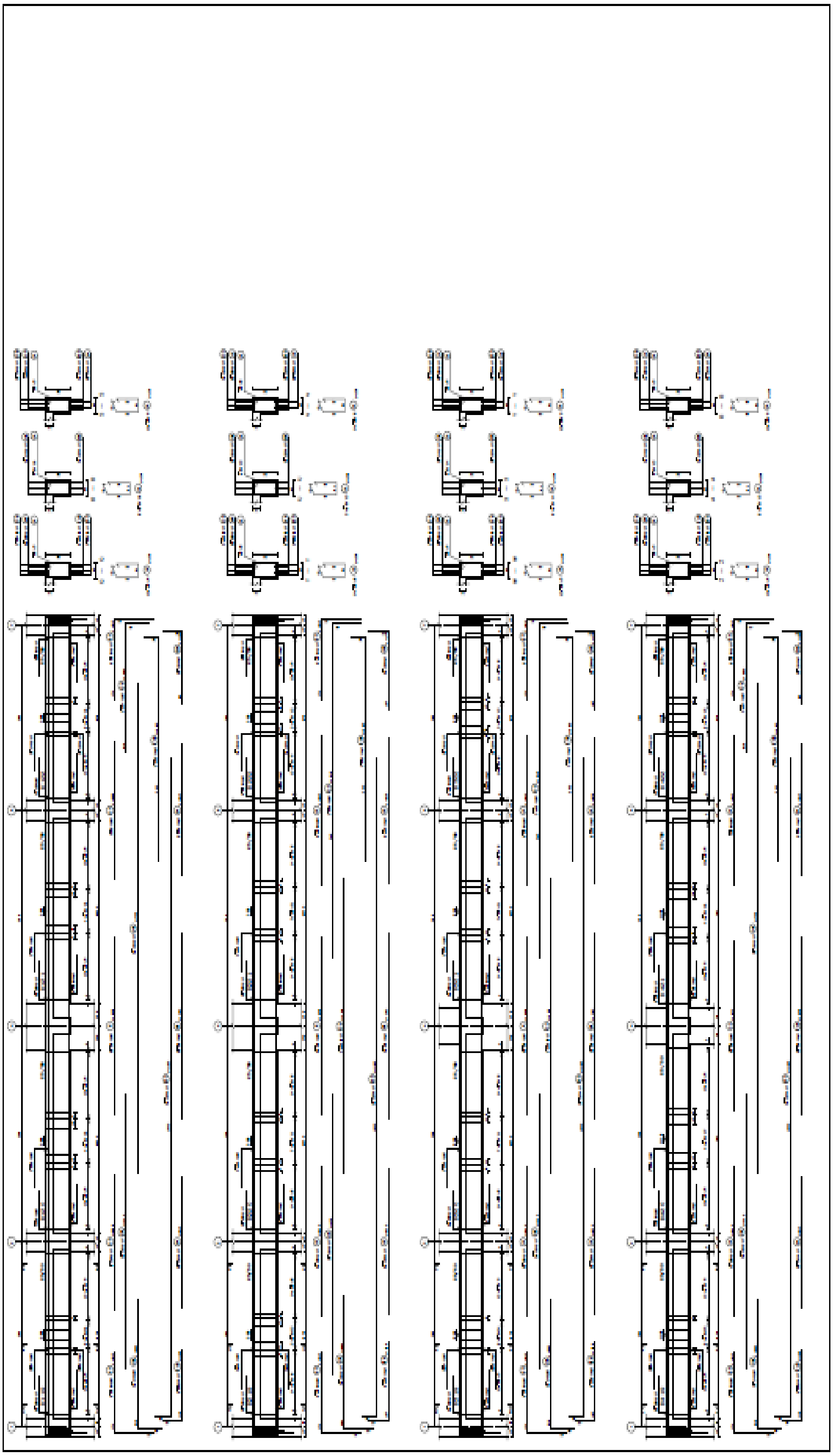

Figure A.66: Beam Details of 8-Story Frame

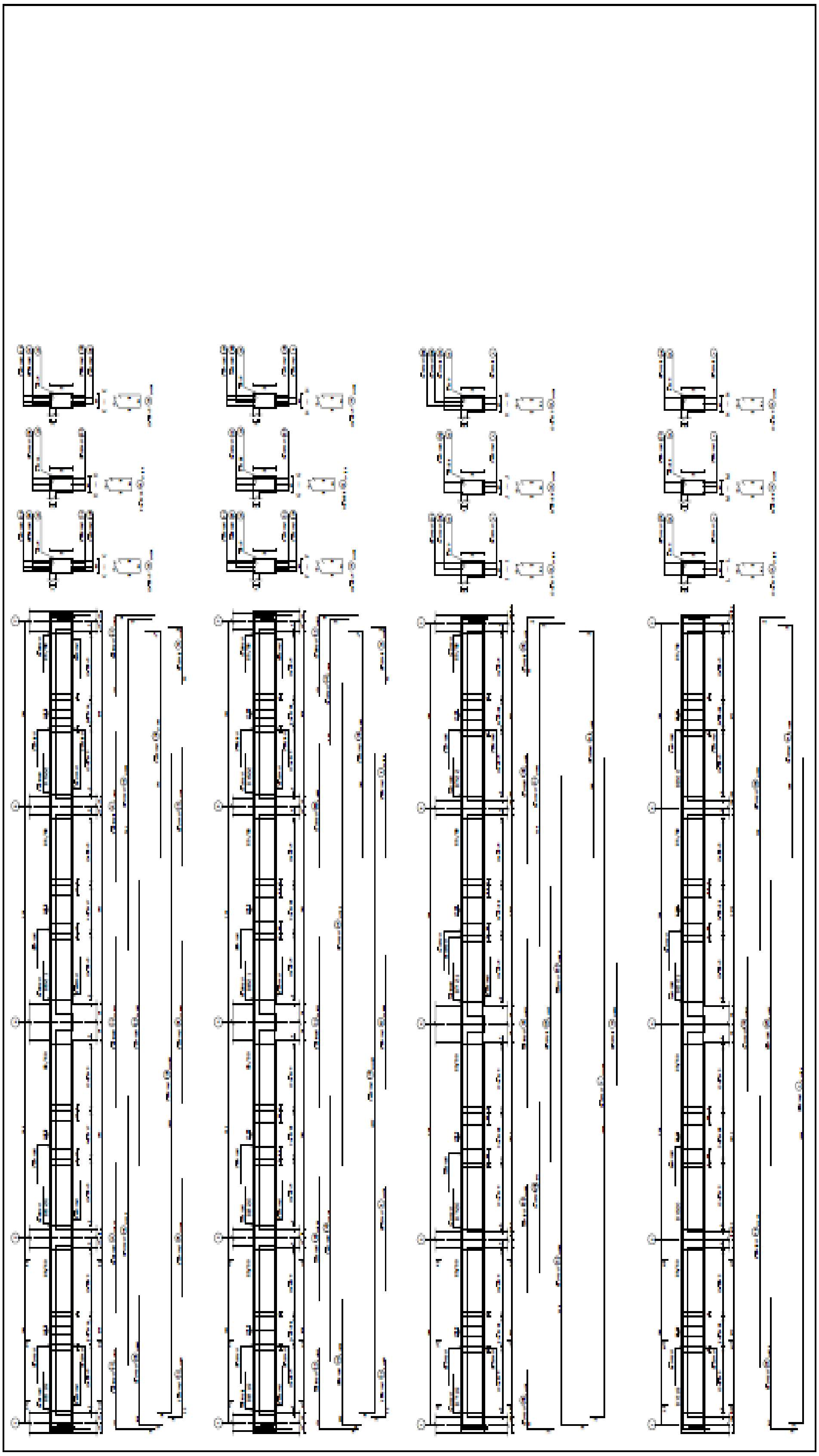

Figure A.67: Beam Details of 8-Story Frame

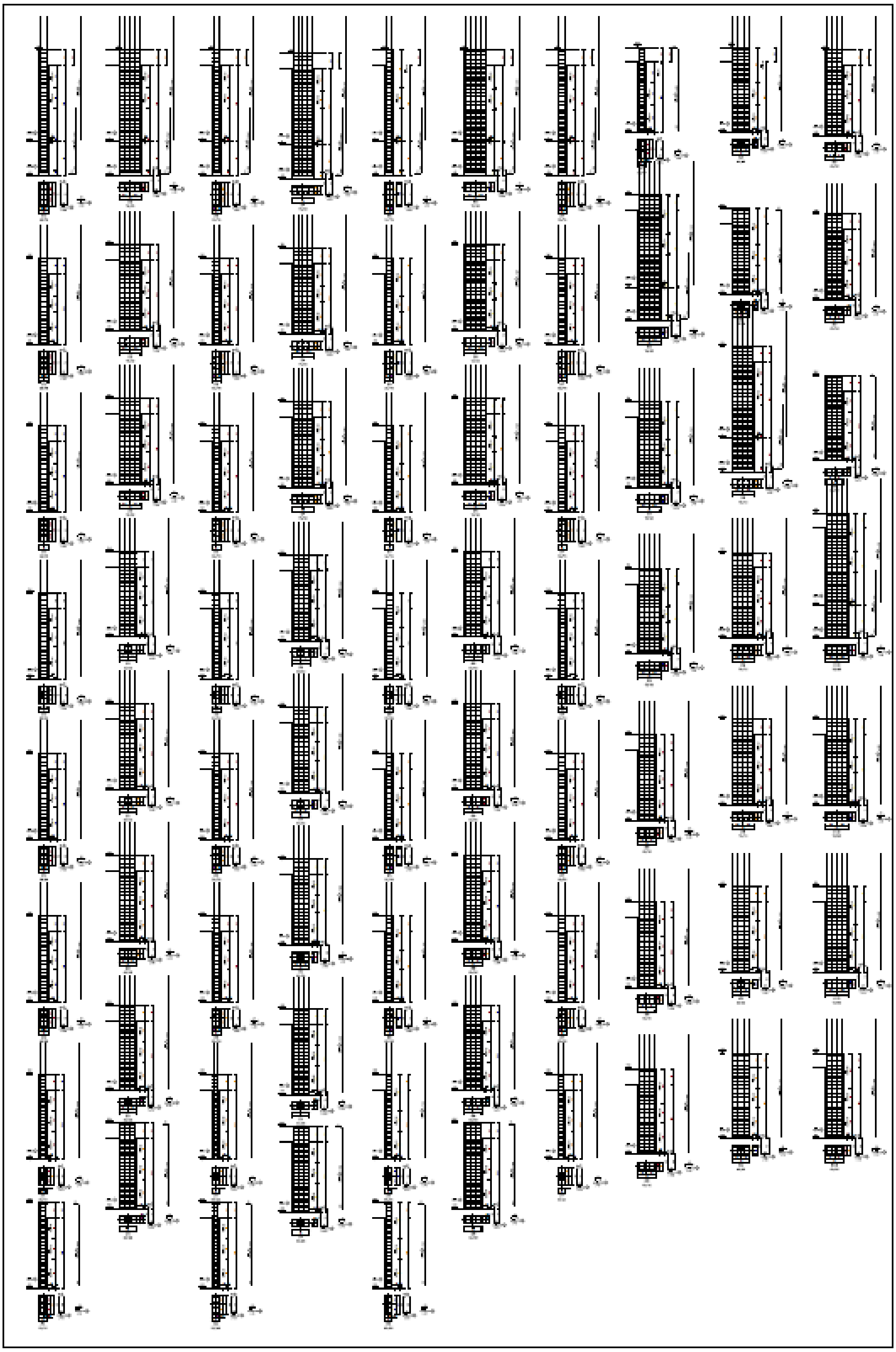

Figure A.68: Column Details of 8-Story Frame

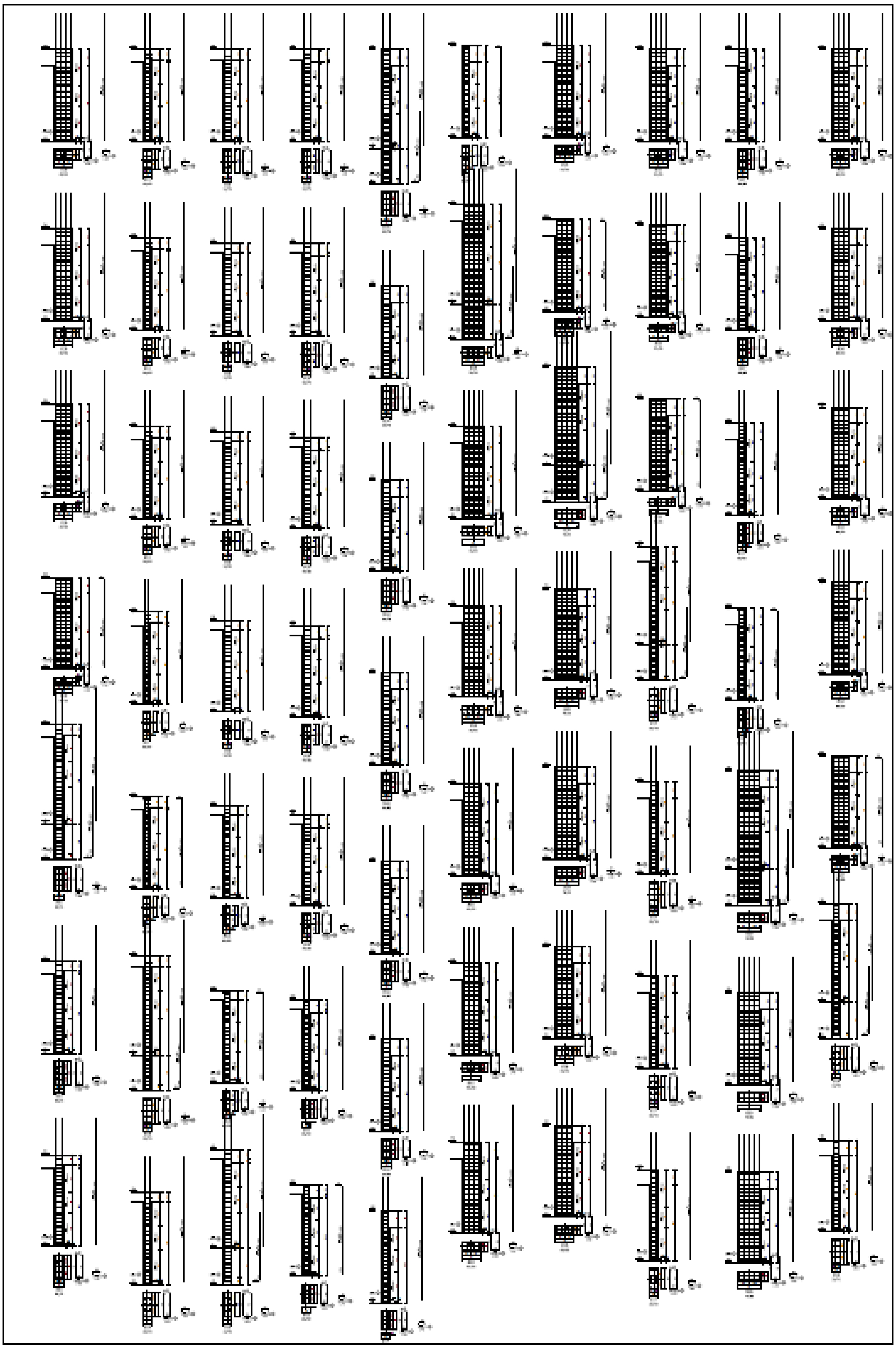

Figure A.69: Column Details of 8-Story Frame

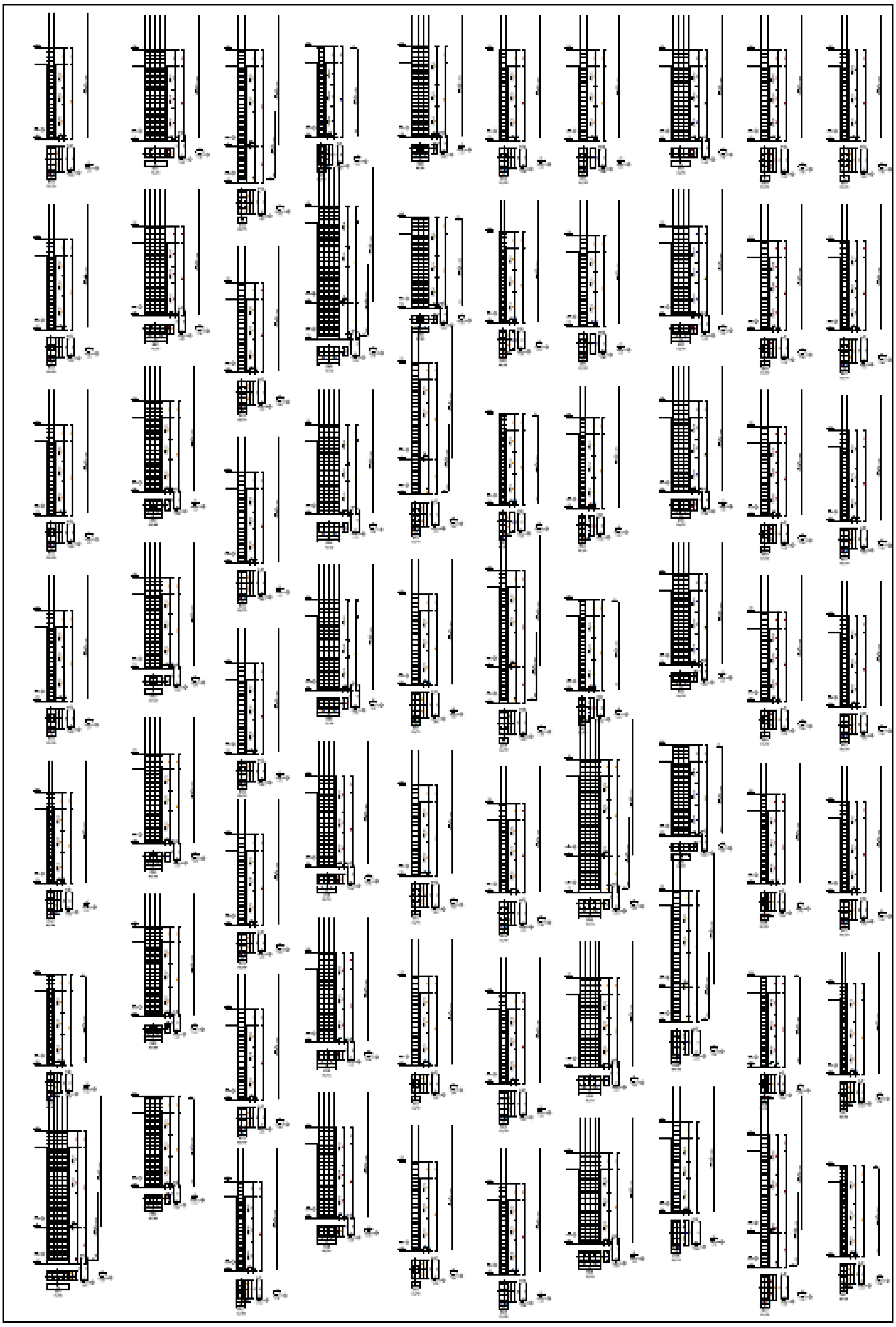

Figure A.70: Column Details of 8-Story Frame